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FOREWORD

The papers in this Circular were all submitted for publication, in substantially their present form, in the Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, held at Rotterdam in June, 1948. They are reproduced here because they may be of interest to engineers who will not have ready access to the conference proceedings.

The papers deal with a variety of subjects, but they have one characteristic in common. With the exception of Dr. Moretto's they record the results of full-scale observations in the field. Dr. Moretto's contribution deals with a laboratory investigation carried out primarily to elucidate certain problems that arose from field observations on a large hydraulic-fill dam. The collection and publication of data concerning the behavior of actual structures, together with adequate descriptions of the soils involved, is urgently needed for the advancement of the arts of foundation and earth dam engineering.

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I. A STUDY OF RETAINING WALL FAILURES

R. B. Peck, H. O. Ireland, and C. Y. Teng

Summary

Under the auspices of the American Railway Engineering Association, questionnaires were sent to the chief engineers of all of the American railways requesting information about retaining walls or abutments that had failed or that had experienced progressive movement to an undesirable extent.

The questionnaire brought forth information about a large number of retaining walls and abutments. The failures and movements have been analyzed with respect to their probable causes. It has been found that, with few exceptions, the difficulties were due to misjudgment of foundation conditions rather than to incorrect assumptions regarding the backfill pressure.

Introduction

In 1945, the Committee on Masonry of the American Railway Engineering Association appointed a subcommittee on earth pressures against masonry structures. The subcommittee's principal assignment was to study and revise the current specifications of the Association with respect to retaining walls and abutments.

As one of the first steps in this study a questionnaire was sent to all principal railroads of the United States and Canada to obtain information about retaining walls and abutments that had performed unsatisfactorily — in particular, about retaining walls that had failed completely or that had experienced movements of such magnitude as to impair their function. Data were not requested about failure of abutments by scour.

The chief engineers of 77 railroads were invited to contribute information. Thirty-seven (48 per cent) did not reply. Twenty-four (31 per cent) reported that no difficulties of any consequence had come to their attention or that failures had occurred only in very old walls designed according to rules of thumb now considered obsolete. The remaining 16 (21 per cent) reported that the behavior of at least some walls and abutments had been unsatisfactory enough to cause concern. These 16 submitted information about 44 walls and abutments that were considered unsuccessful. The location of these structures is shown in Fig. 1.

Inasmuch as almost 80 per cent of the individuals to whom questionnaires were sent either reported no difficulties or did not reply, it would appear a reasonable conclusion that the great majority of retaining walls and abutments can be considered successful and that as a rule the present methods of design are at least adequate and possibly conservative. Nevertheless, a sufficient number of failures or of examples of excessive movement was reported to indicate that walls designed according to the customary procedures are not necessarily stable or static.

Description of Walls and Movements

The unsatisfactory walls and abutments are classified in Fig. 2a according to their height, and in Fig. 2b according to the magnitude of the movement they experienced. The majority experienced forward movements of 6 to 12 in. This is probably an indication that movements less than 6 in. are commonly not a matter of concern, and suggests that movements smaller than about 3 in. are usually considered quite normal and satisfactory.

In Fig. 3 the walls are classified according to the type of movement they experienced. It may be observed that about 18 per cent failed completely. That is, they overturned, or broke structurally, or were considered in such imminent danger of collapse that they had to be

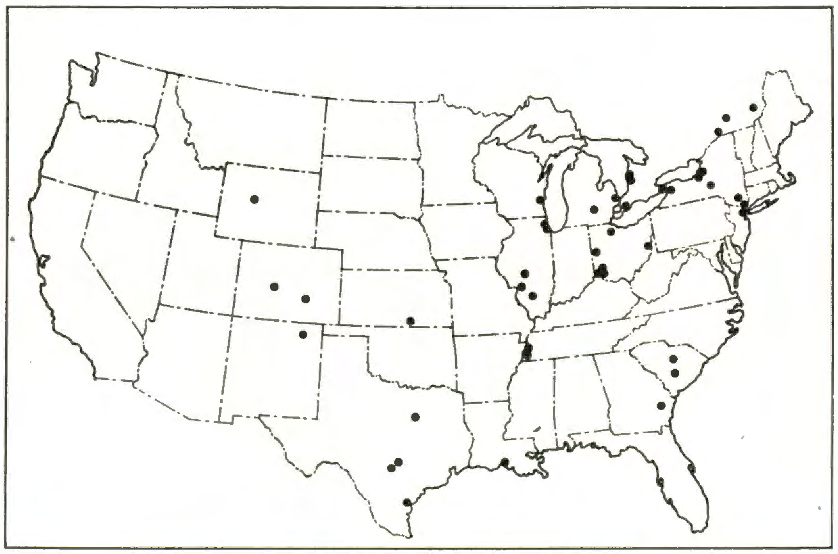


Fig. 1. Location of Unsuccessful Walls and Abutments

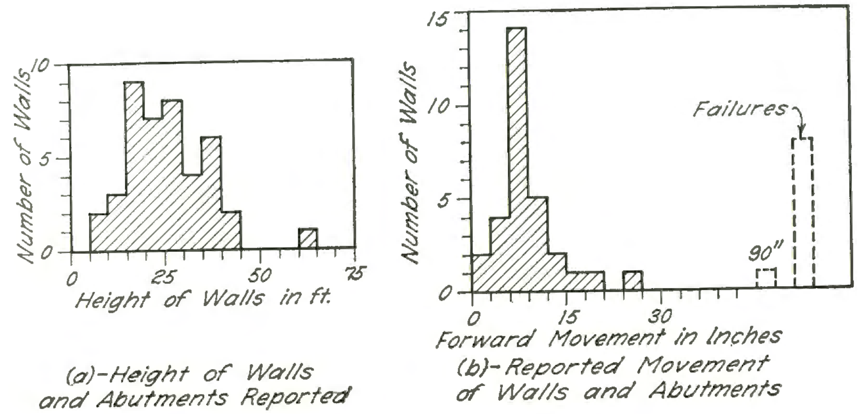


Fig. 2. Classification of Unsatisfactory Walls and Abutments

either strengthened or else removed and replaced. More than half the walls experienced a progressive outward or tilting movement.

Figure 3 also indicates that almost half of the unsatisfactory walls were supported on piles. This fact suggests that there was general recognition of unsatisfactory foundation conditions and that an attempt to improve the foundations was made by providing pile support. Nevertheless it does not appear that the mere use of piles, even including batter piles, sufficed to prevent excessive movement of the walls.

Figure 4 shows the foundation and backfill materials associated with those walls that experienced progressive outward or tilting move-

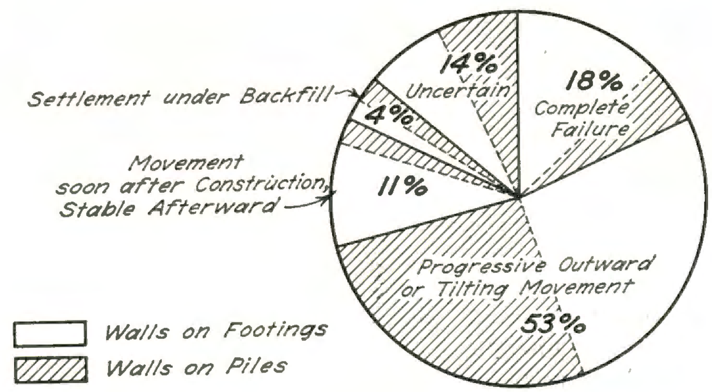


Fig. 3. Classification of Walls According to Type of Movement

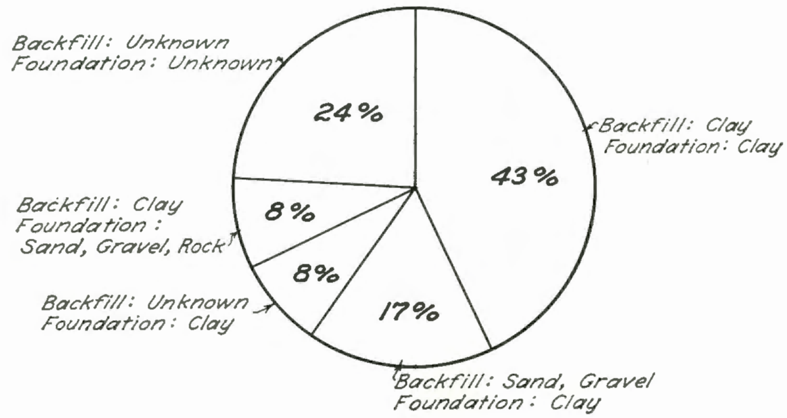


FIG. 4. FOUNDATION AND BACKFILL MATERIALS OF WALLS THAT EXPERIENCED PROGRESSIVE OUTWARD OR TILTING MOVEMENTS

ments. In every reported example of progressive movement where there was sufficient information to classify the soils, clay occurred in the foundation or in the backfill, and in most instances occurred in both. This suggests that present and past design procedures must be quite conservative when foundation conditions are good and when the backfill consists of sand or gravel. On the other hand, it also suggests that the factor of safety of walls founded on clay or backfilled with clay is probably on the average much lower than believed by the designers.

Typical Examples of Unsatisfactory Behavior

The first example of unsatisfactory behavior is illustrated in Fig. 5, which shows the pertinent data concerning one of a pair of open abutments constructed for a grade separation. It was recognized that the foundation for the abutments would require pile support, and the structure was founded upon cast-in-place concrete piles having an embedment of 45 ft. One group of four of these piles was tested, with the results shown in the figure. On the basis of the test it appears that the average shearing resistance of the soil was approximately 0.17 ton per sq ft.

To lighten the load on the abutments as much as possible and at the same time to reduce the active earth pressure of the railroad embankment, the abutments were made hollow and it was specified that the fill should slope downward toward the toe. In spite of these precautions, each abutment began to settle during construction, and when the total dead load reached 5000 tons each abutment began to

tilt toward the fill and to sink into the ground. At this stage the average settlement increased very rapidly from 4 in. to almost 10 in. Thereupon the fill was removed and the design altered in such a way that no fill rested upon the bases of the abutments when the bridge was completed.

On the assumption that the average shearing resistance of the soft subsoil was 0.17 ton per sq ft, the total ultimate bearing capacity

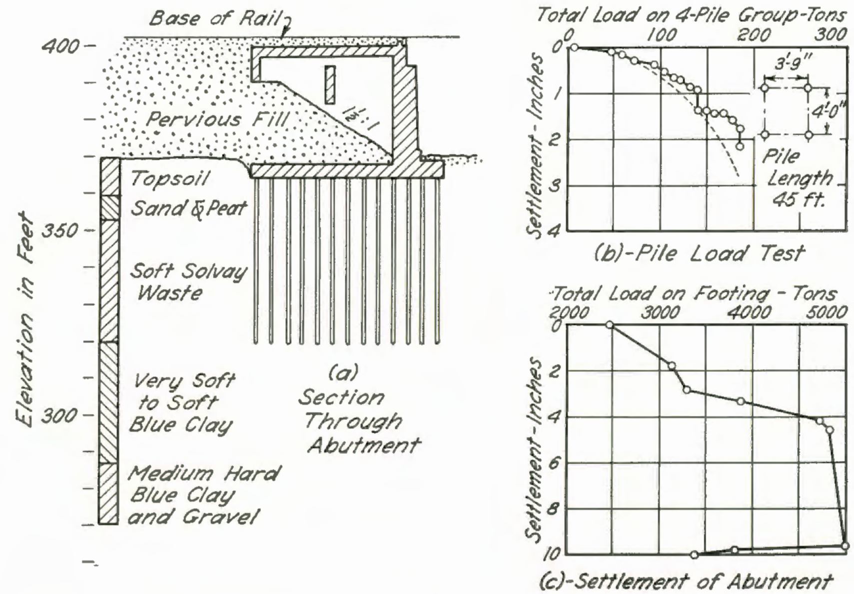


FIG. 5. UNSATISFACTORY BEHAVIOR: DATA CONCERNING ONE OF A PAIR OF OPEN ABUTMENTS CONSTRUCTED FOR A GRADE SEPARATION

of the base of the abutment should have been approximately 5300 tons. This value is in reasonable agreement with the observed load at failure. Therefore it appears obvious that the failure of the abutment was caused by overloading the subsoil and had little if any relation to the active earth pressure against the abutment.

Figure 6 shows the pertinent data concerning a typical example of progressive forward movement of large magnitude. The piles penetrated the soft material and rested on the medium clay at a depth of 40 ft. The two abutments and two intermediate piers for the bridge were completed, and backfilling was in progress, when movements of all four elements of the substructure were observed. Measurements

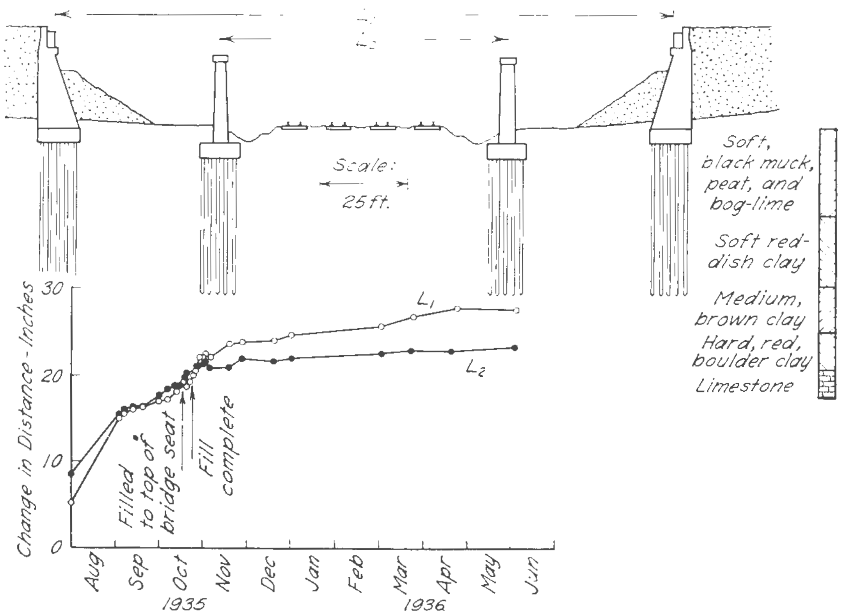


FIG. 6. UNSATISFACTORY BEHAVIOR: DATA CONCERNING A TYPICAL EXAMPLE OF PROGRESSIVE FORWARD MOVEMENT OF LARGE MAGNITUDE

yielded the results indicated on Fig. 6. The distance L_2 between the intermediate piers decreased almost as much as the distance L_1 between abutments. This means that the movement was deep-seated and involved the lateral squeeze or flow of clay toward the center of the bridge from each end. Such a movement could hardly be caused otherwise than by overloading the clay stratum by the weight of the backfill. Under the weight of the fill the bearing capacity of the soft reddish clay and overlying material was probably approached and a slow lateral flow or creep was initiated by the excessive shearing forces. The lateral pressure against the abutment was probably not excessive, because the abutments were of the open type and the fill was allowed to extend through them.

This example indicates that the lateral forces acting against a vertical section through an abutment supporting a high fill may be very great — considerably greater than the forces due to the active earth pressure against the abutment itself. This fact was demonstrated clearly by the behavior of a structure built during the war to store iron ore. The structure consisted of two parallel retaining walls

about 30 ft high and 275 ft apart. The walls were tied to each other by a series of steel rods just below the ground surface. A deep deposit of medium clay underlay the structure. The behavior of the walls was observed by means of strain observations on the tie rods. At the yield point of the rods, their capacity to resist the horizontal forces against the retaining walls was 70,000 lb per linear ft of wall. When the iron ore was piled to a height of 22 ft the rods reached their yield point strain. Yet according to any rational method of computation the active earth pressure of the ore against the walls could not have exceeded about 16,000 lb per linear ft. Therefore the actual horizontal forces exerted against the walls and their foundations were more than four times the computed earth pressures. Independent observations demonstrated that the ultimate bearing capacity of the clay beneath the storage yard was 2.6 tons per sq ft, whereas the weight of 22 ft of ore was 1.8 tons per sq ft. Therefore the factor of safety against a bearing capacity failure was only about 1.4. At such a low factor of safety, excessive and continuous horizontal deformations in the subsoil are to be expected.

In a large number of the other examples of progressive outward movement or tilting, conditions appeared to be similar to those indicated in Fig. 6 except that the movements were generally much smaller. This would suggest that the shearing stresses in the foundation were considerably smaller with respect to the shearing strength of the soil than in the example described. Nevertheless, it is believed that in practically all the examples the lateral forces in the subsoil of the structure were considerably greater than the computed active earth pressure. Progressive movements were observed even on several walls supported by fairly stiff clays.

In a number of instances, progressive movements seemed to have begun during the 1930s in spite of the fact that the walls had been apparently static for many years before. This was generally attributed by the railroad engineers to the marked increase in locomotive weights during this period. Many of the walls were designed for Cooper's E-30 or E-40 loading, whereas by the 1930s the weight of motive power had generally increased the loading to E-60 or E-70. It is quite possible that this increase in live load, with accompanying increase in toe pressures, was responsible for the beginning or revival of movements. There is, however, no satisfactory way to evaluate the relative importance of this factor.

In most instances where both the backfill and the foundation consisted of clay, it was not possible to ascertain the relative importance of those movements associated with overloading the clay foundation and those that may have been caused by the progressive decrease of the strength of the backfill material. The evidence appears to indicate that foundation failures or foundation movements were more prevalent than those caused by an increase in the backfill pressure of clay. However, several of the complete failures undoubtedly belong in the latter category, and it is certain that clay backfills did in several instances exert an increasing pressure.

In two examples concrete gravity walls were built to retain clay fills sloping at 1½ to 1 upward and away from the crest of the wall. In one case the top of the fill was about 8 ft above the top of the wall, and the wall itself was 8 ft high. In the other the wall was only 6 ft high and the fill was about 60 ft high. Both walls were stable for a number of years. Both failed structurally at a point above the foundations. This fact indicates that deep-seated foundation movements could not have been the primary cause of failure. Hence it must be inferred that the pressure of the clay gradually increased until failure occurred.

Conclusions

The records obtained by means of the questionnaire are still being studied and analyzed. However, it is believed that several rather definite conclusions can be drawn at present.

1. Considering the total number of conventionally designed retaining walls and abutments, failure or progressive movement is relatively uncommon.

2. Unsatisfactory behavior is rarely encountered unless the subsoil of the wall, or its backfill, or both, contains clays or clay-like materials. The most prevalent cause of trouble is the overloading of clay foundations by the weight of the backfill. The second most common cause is probably the gradual increase of pressure when the backfill material consists of clay. A possible third cause of some importance is the increase of live load.

3. Contrary to the opinion of some engineers, it seems doubtful that the effects of vibration due to traffic on granular backfills are of serious importance. Otherwise it is probable that more difficulties would have been reported with walls backfilled with sand.

4. The unsuccessful behavior of walls has been due primarily to misjudgment of the foundation conditions. Present methods of design place the emphasis almost exclusively on computed earth pressures

that often have little relation to the real forces that the structures must resist. In the future, much more attention should be given to the foundation conditions and to possible time-conditioned changes that may occur in the backfill. The theoretical study of earth pressures has provided a fascinating diversion for engineers, but it has tended to blind them to an understanding of the real behavior of earth-retaining structures.

II. EXPERIENCE WITH FLEXIBLE CULVERTS THROUGH RAILROAD EMBANKMENTS

O. K. PECK AND R. B. PECK

Introduction

About 1926, installation of large-diameter flexible steel culverts was begun on the Denver & Rio Grande Western Railroad in Colorado, Utah, and New Mexico. About thirty such culverts ranging in diameter from 7.5 to 15 ft were placed beneath fills varying in depth from 2 to 50 ft. The behavior of these culverts has been closely observed since that time, deflection measurements have been made on a number of the structures, and measurements have been made in detail on two that were subjected to extreme conditions of backfilling. This paper describes the results of the observations.

General Discussion

The culverts consist of corrugated steel or iron plates bent to a circular shape before delivery and assembled into a cylindrical unit in the field. The thickness of the steel varies from 0.1719 in. to 0.2812 in. The corrugations are spaced at 6-in. intervals; the depth of the corrugations is $1\frac{1}{2}$ in. Hence the pipes are relatively flexible.

After the pipes have been erected they are backfilled with selected material. The soil close to the pipe, especially near the bottom, is thoroughly tamped by hand and all the material beside the pipe is carefully compacted in layers. During the process of backfilling, vertical struts are located in the pipes, their length being such that the vertical diameter exceeds the horizontal diameter by about 3 per cent. At the upper end of each strut is placed a block of soft wood so compressible that it will crush and permit a decrease of diameter as the vertical load on the culvert gradually increases. After the fill has settled and a stable condition has been reached the struts are removed.

Flexible culverts of this type depend upon the resistance of the surrounding soil for their stability. The thickness of the metal shell is determined by the requirement that sufficient bearing must be provided for the bolts in the longitudinal joints between segments to withstand the circumferential compression produced by the weight of the overburden. Bending moments due to differences in the intensity of vertical and lateral pressure are ignored. It is assumed that the culvert will deform sufficiently to develop pressures practically equal in all directions around the entire structure. The stability of the shell therefore requires only that the changes in diameter be within reasonable limits.

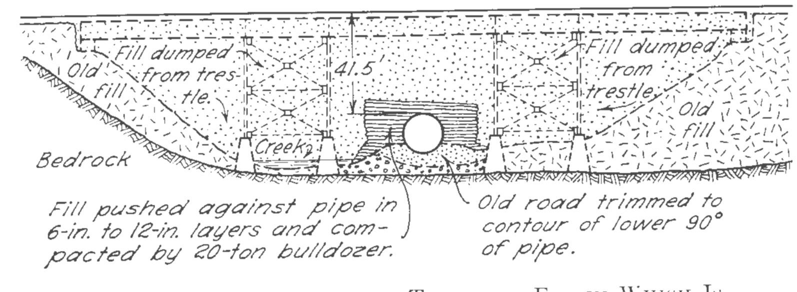


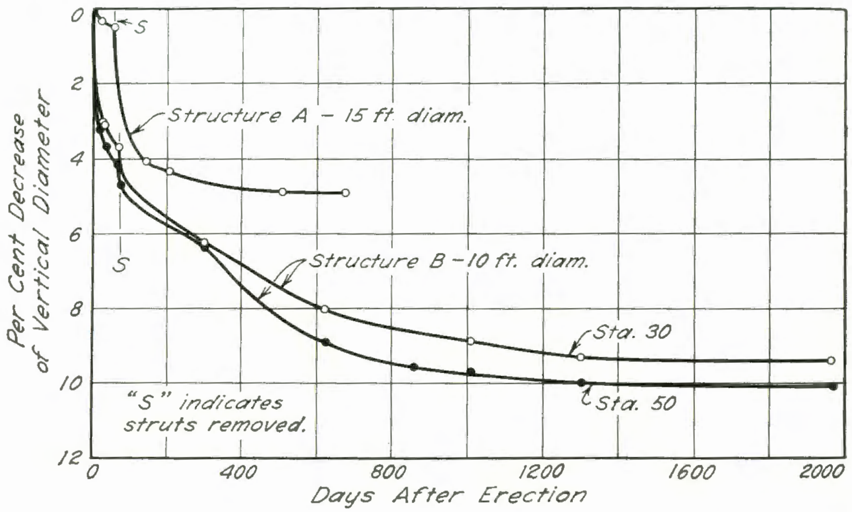
FIG. 1. LONGITUDINAL SECTION THROUGH A FILL IN WHICH IS BURIED A PIPE CULVERT 15 FT IN DIAMETER

Figure 1 shows a longitudinal section through a fill in which is buried a pipe culvert 15 ft in diameter. The depth of the cover is 41.5 ft. The base of the culvert rests in a depression trimmed to the contour of the plates through the lower 90 deg of the structure. The depression was carved in an old wagon and automobile road built on a subgrade of disintegrated granite ranging in size downward from 1 in. to particles of silt diameter. On either side of the rest of the culvert a fill consisting of disintegrated granite was pushed against the pipe in 6-in. to 12-in. layers and compacted by a 20-ton bulldozer. Above the top of the pipe, the fill was placed by dumping from a trestle. All backfill material was granular. Near the pipe, no pieces larger than 3 in. in diameter were permitted.

The relation between time and the shortening of the vertical diameter of this culvert is shown in Fig. 2 by the curve marked Structure A. A small amount of crushing took place while the struts were still in position. At the time indicated by S, the struts were removed and the vertical diameter shortened rapidly for a few weeks. However, at the end of 200 days, movements had virtually ceased, and no appreciable movements could be observed after 500 days. The lengthening of the horizontal diameter was approximately equal to the shortening of the vertical diameter.

The material surrounding this culvert was resistant to deformation, and the diameter changes were moderate. On a second structure, designated as Structure B, 10 ft in diameter, much less favorable material was available for both the foundation and the backfill. Indeed Structure B represents the most unfavorable conditions under which any of the culverts were installed.

Figure 3a shows a cross-section through the fill in which Structure B was located. The depth of cover was 13 ft. A cross-section of the culvert is shown in Fig. 3b. This diagram shows the soil conditions



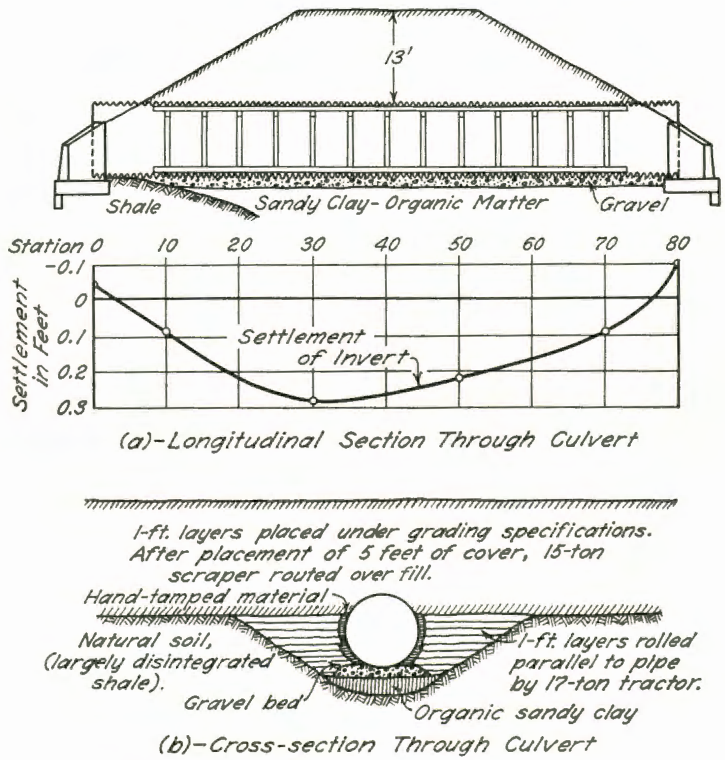


Fig. 2 (Above). Relation Between Time and Shortening of Vertical Diameter
Fig. 3. Settlement of the Culvert

and the method of compaction. Except at the upstream end where the culvert rested upon shale bedrock, the structure rested on sandy clay with considerable organic matter. A bed of gravel about 18 in. thick was placed in a shallow trench excavated in this material, but as much as 4 ft of relatively compressible clay remained beneath the gravel bed. The backfill consisted of a residual silty sand derived from shale and contained numerous rather large fragments of unweathered shale. The effective size was about 0.25 mm and the uniformity coefficient 20. The liquid limit of the material passing a No. 28 mesh sieve was 44.1 per cent and the plastic limit 28.8 per cent. Hence the binder material might properly be classified as a silt of medium compressibility.

The ends of the culvert were protected by concrete headwalls as shown in Fig. 3a. The concrete headwalls were cast directly against the pipe and were bonded to it by bolts.

The decrease of the vertical diameter at Stations 30 and 50 near the middle of the culvert is shown in Fig. 2. It is observed that the percentage diameter change equals approximately twice that for Structure A and that about three years were required for the structure to reach equilibrium. Although the performance of this structure was not as satisfactory as that of any of the other culverts, the structure nevertheless developed no signs of failure and was entirely adequate for its purpose.

On account of the compressible material beneath the fill the base of the fill settled appreciably. The settlement of the invert of the culvert during the first three months is shown in Fig. 3a. The settlement placed the bottom of the culvert in axial tension. The only visible effects of the tension, however, were a few small cracks in the headwalls and a relative movement indicating that the upper part of the culvert was pulling out of the headwalls. At a similar site close to the location of the flexible culvert a concrete box culvert was established. It was subjected at the bottom to tension of the same nature as that experienced by the flexible culvert, and as a consequence developed a transverse crack across the invert and part way up the sides.

The behavior of the other flexible culverts on which measurements were made is indicated in Table 1. The movements are generally intermediate between those of Structures A and B. Several of the culverts, including one 10 ft in diameter under 3 ft of cover, were subjected to heavy traffic with the largest locomotives currently in operation. No structural defects of any consequence have appeared.

Table 1
Observed Distortion of Flexible Culverts at Center of Fill

Diameter, ft	Metal Thickness, in.	Overburden, ft	Years Since Erection	Percentage by Which Vertical Diam Is Less Than That of True Circle
7.5	0.1875	2	3	−2.5
7.5	0.1875	4	3	2.7
7.5	0.2188	9	3	1.4
8.75	0.2188	4	3	1.7
8.75	0.2188	5	4	1.7
8.75	0.2188	9	3	4.7
8.75	0.2188	12	3	0.0
8.75	0.2188	13	3	3.2
8.75	0.2188	15	3	1.9
10.0	0.2188	3	2	1.3
10.0	0.2188	3	6	4.3
10.0	0.2188	6	3	0.3
10.0*	0.1875	13	5	7.1
10.0	0.2813	19	2	2.8
10.0	0.2813	19	2	1.8
10.0	0.2500	46	3	2.3
10.0	0.2813	50	4	5.1
15.0†	0.2813	37	2	0.1

Note: Total vertical diameter change approximately 3 per cent plus value given in table.
* Structure B.
† Structure A.

In contrast to the culverts described above, a similar installation was made on a portion of roadbed subsequently acquired by the Denver & Rio Grande Western. The culvert had been placed on the bottom of a ravine crossed at a high level by a wood trestle. The trestle was replaced by a granular fill. The lower part of the fill consisted of frozen chunks pushed in by a bulldozer. The rest was dumped from the top of the trestle. No serious efforts were made to compact the material around the pipe, and several large voids were left between the timbers of the trestle and the pipe itself. During subsequent operations of the railroad the structure of the loose fill collapsed and a locomotive was derailed when the roadbed gave way. The operations necessary to reconstruct the culvert were costly.

Summary and Conclusions

As a result of experience, it is believed that flexible culverts when carefully installed and properly backfilled represent safe and economical structures which require less maintenance than rigid culverts of the same size. Chiefly on account of the ease of installation, their initial cost is much less than that of other types providing comparable drainage openings. However, since flexible culverts possess very little inherent strength of their own, it is essential that backfilling

operations be carefully supervised and controlled. Because of the difficulty of obtaining adequate compaction from the bottom to about the lower third-point of the height of the culverts, it would appear advisable to prepare a bed of compacted fill and to trim it to the contour of the lower part of the culvert with the aid of a template. Except for culverts backfilled with highly plastic soils, with which there has been no experience on this railroad, it is believed that the preceding conclusions have general applicability.

The effect of dissolved substances in the groundwater deleterious to steel or iron should be considered. However, in many parts of the western United States concrete culverts are generally subject to more rapid deterioration than galvanized steel or iron.

The foregoing discussion makes it clear that theoretical analyses based on the results of soil tests are not warranted in connection with the design of flexible metal culverts. If the soil is adequately compacted, a moderate deformation of the culvert will establish a state of nearly uniform all-around pressure. As a consequence, the section modulus of the culvert plates is practically irrelevant. If the culvert metal and the bolted joints are adequate to resist the ring stresses, no other stress conditions need be investigated. Field supervision of backfilling operations is, on the other hand, of outstanding importance.

III. SETTLEMENT OBSERVATIONS ON A LARGE WATER TANK, SALT LAKE CITY, UTAH

O. K. Peck and R. B. Peck

Summary

This paper describes the results of settlement observations on a water storage tank located above a deep bed of stratified clay and sand. The relationship between settlement and time is discussed. In addition, a comparison is made between the observed settlements and those computed on the basis of a simple statistical relationship between the compressibility and the liquid limit of normally loaded clays.

History of Structure

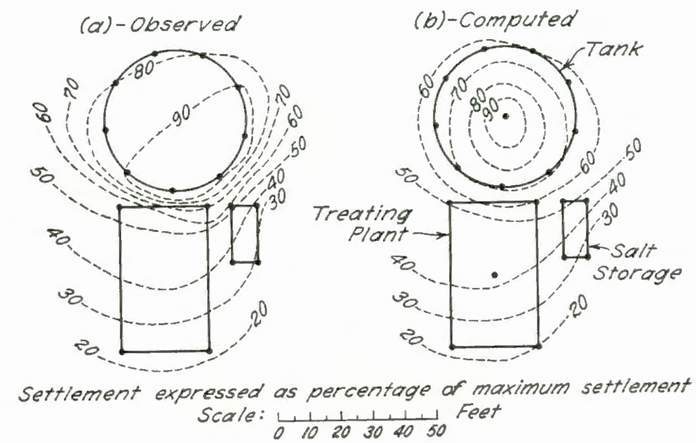
The water storage tank is one of a group of structures comprising an installation for the treatment of water for use in locomotives. Also included in the group are a one-story treating plant 47 by 25 ft in plan, and a structure, 21 by 9 ft in plan, used for salt storage. The general relationship between the three structures is shown in Fig. 1. At the closest point the treating plant is 5 ft from the foundation of the tank.

The storage tank has a diameter of 35 ft and a height of 60 ft. It rests upon a circular reinforced concrete mat 45 ft in diameter. The mat in turn rests upon a compacted gravel fill about 6 ft thick. The general arrangement of the tank foundation is shown in Fig. 2.

Before the construction of the foundation for the tank, 5 borings were made with a post-hole auger to a maximum depth of 15 ft. The borings indicated the presence of as much as 10 ft of compact fill placed a number of years before. Between depths of 10 and 15 ft a deposit of loose sand was found. A sample taken from a depth of 15 ft indicated that the material was fine-grained and extremely uniform.

On account of the loose character of the sand, the mat upon which the tank was founded was heavily reinforced. For the same reason the treating plant itself was supported on a short-pile foundation. The piles were intended merely to develop enough skin friction in the fill and sand to carry the relatively small weight of the structure without excessive settlement.

The treating plant and tank were completed on January 27, 1942. In the expectation that there might be a small amount of differential settlement which would necessitate restoring the tank to a vertical position, nine reference points were spaced equally around the periphery of the concrete mat, and the elevation of these points was deter-



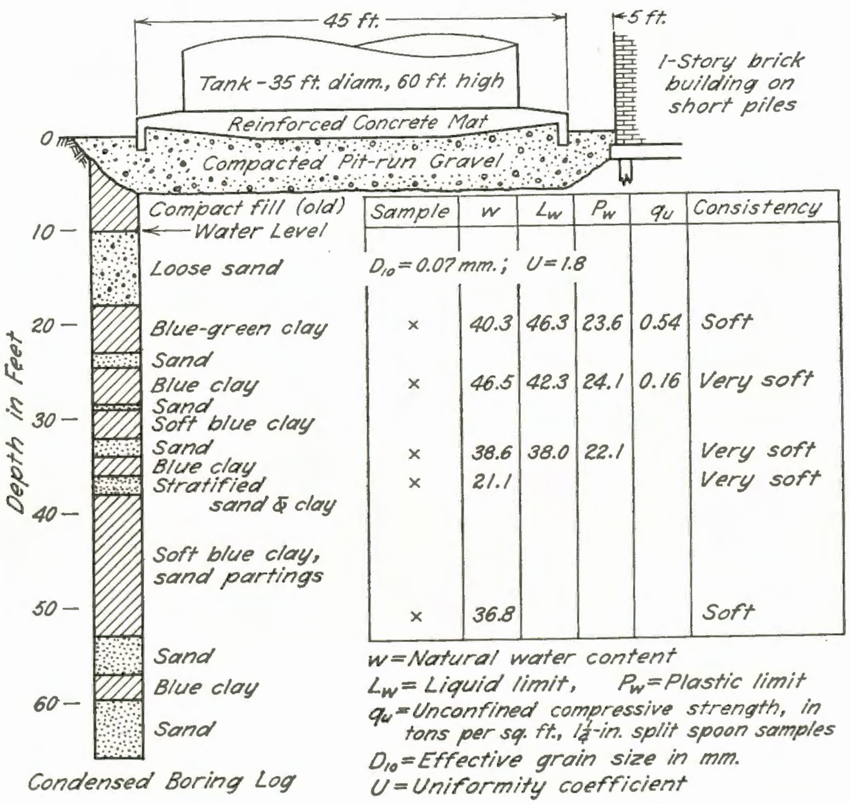


Fig. 1 (Top). Relationship Between Tank, Treating Plant, and Storage Structure

Fig. 2. General Arrangement of Tank Foundation

mined with respect to two independent benchmarks located on opposite sides of the tank at distances of 82 and 93 ft from the center line of the tank.

Four days later, on January 31, the tank was filled with water in order to test the structure. During the process of filling, the average settlement of the tank reached a value of 0.4 ft. Two days later the settlement had increased to 0.49 ft and after a week to 0.58 ft, or approximately 7 in. Because of the progressive nature of the settlement, readings were taken daily until February 22, twenty-three days after the first filling.

By this time the settlement of the tank exceeded 0.7 ft and it became evident that the movement was of deep-seated origin, inasmuch as settlements of this order of magnitude could not be attributed to the sand disclosed by the borings. Pending decision as to the proper procedure, the tank was emptied, whereupon it rose slightly less than 1 in. and remained stationary for a 17-day period in the empty condition. The soil exploration equipment of the railroad was brought to the scene and a boring was made. Samples obtained at close intervals by means of a split spoon 1¼ in. in diameter were examined and described by the geologist in charge of the boring equipment. A condensed form of the geologist's report is shown in the boring log on the left side of Fig. 2. At a depth of about 18 ft a deposit of soft clay containing sand layers was encountered. This material extended to a depth of at least 60 ft. The boring ended at a depth of 67 ft in sand. All the clay members of the deposit contained numerous thin partings of sand, many at a spacing of not more than 1 in.

Several samples were preserved in a fairly intact state and were shipped to the laboratory for tests. The results of the determinations of natural water content, liquid limit, and plastic limit are shown in Fig. 2. In addition the unconfined compressive strength of two samples was determined. These two were the only samples sufficiently undisturbed to justify the tests, and it is probable that their strength was appreciably less than that of the natural soil. Nevertheless the results indicated that the clay was of a soft to very soft consistency. The tests on the disturbed samples indicated that the natural water content was approximately equal to the liquid limit. This condition suggested that the clay was part of a normally consolidated deposit. As a matter of fact, the water tank is located within the area defined by the shore line of glacial Lake Bonneville, and the stratified clay and sand are undoubtedly a lacustrine deposit formed in this ancient predecessor of the present Great Salt Lake.

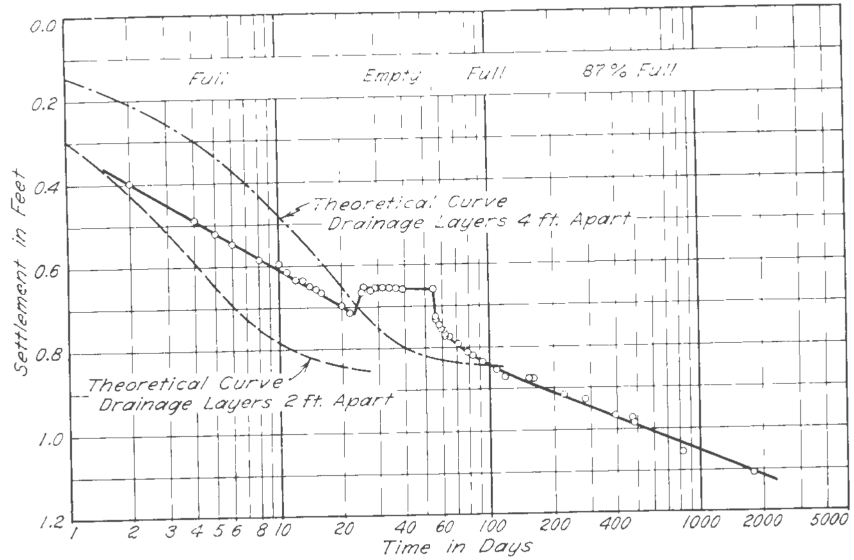


FIG. 3. RESULTS OF STUDIES OF SETTLEMENT LEVELS

At the same time that the borings were being made the results of the settlement levels were being studied. They are shown in Fig. 3, in which the average settlement is plotted as a function of the logarithm of time. It was observed that up to the time the tank was emptied the points representing the settlements fell upon a straight line. The reasons for this empirical fact were not clearly understood but it was considered improbable that the settlement should accelerate in such a manner that future points would fall below the line defined by the observations. If the trend continued at its original rate for a period of as much as 100 years, the average settlement would not exceed 1½ ft. Since this amount could be tolerated by the pipe connections between the tank and the treating plant it was decided to refill the tank.

As shown in Fig. 3, the tank was again filled 56 days after construction. It remained full until approximately 200 days. At this time the treating plant was placed in operation. Under operating conditions, the tank is approximately 87 per cent full. Correspondingly, the slope of the settlement–log time curve after 200 days was flatter than during the first 23 days. The latest readings taken in 1946 continue to indicate that the plotted relationship approximates a straight line and suggest that the 100-year settlement will be somewhat less than the value given above.

Discussion of Rate of Settlement

Part of the settlement that occurred when the tank was filled undoubtedly had its origin in the loose sand beneath the gravel fill; a very small part may have originated within the gravel fill itself. An estimate of the magnitude of the settlement due to the gravel base and loose sand can be made by considering the behavior of a very similar tank on a nearly identical foundation at Salida, Colorado. This tank is 32 ft in diameter and 50 ft high. The compacted base rests on fine loose sand and some coarser material, all of which has been deposited by the Arkansas River on its bedrock valley floor. No clay underlies the tank. On its first filling, the average settlement of the tank was 0.19 ft. After several fillings the settlement increased to 0.24 ft.

The tank in Salt Lake City is somewhat larger and exerts a somewhat greater total load on its foundation. Therefore it seems reasonable to assume that a settlement of at least 0.20 ft can be attributed to the cohesionless material above the clay. This represents half of the total settlement observed 2 days after the tank was first filled.

The settlement–log time curve, Fig. 3, appears to be linear between 2 and 23 days after the tank was filled. After the readjustments due to emptying and refilling the tank, the curve again appears to be linear. Hence the plotted curve consists essentially of two straight lines that differ in slope, possibly because of the reduction in load after about 200 days.

The dash curve in Fig. 3 shows the theoretical time-settlement relation computed on the assumptions that the entire weight of the structure was applied to the subsoil on the date the tank was filled, and that the effective spacing of the drainage layers is 2 ft. The dash-dot curve shows the relation for a spacing of 4 ft. If the drainage layers were spaced at 0.6 ft, which is quite possible insofar as the boring log is concerned, the entire computed primary settlement of 0.87 ft would have required only about 2 days.

It is apparent that no reasonable assumptions regarding the drainage conditions lead to a theoretical time-settlement curve resembling the one based on the observations. The reasons for the difference are not fully understood. However, one factor is probably the rate of load application. Placement of the gravel fill increased the soil pressure on the area to be covered by the mat by 450 lb per sq ft, and construction of the mat and tank increased the pressure to 1020 lb per sq ft. These operations took place over several months, and whatever settlement occurred during this period was unobserved. However, the water load while the tank was being tested was rapidly applied and it amounted to 2250 lb per sq ft. The observed settlement–log time curve represents the combination of the complete curve due to the rapidly applied load and the latter part of the curve due to the preceding gradually applied load. If the curve for the preceding loads had already passed its point of contraflexure and was concave upward when the water load was applied, the two simultaneous processes could have produced a settlement-log time curve that was almost straight during a limited period of time. Under this interpretation, a period of 2 or 3 months would have been required for primary consolidation, but this can hardly be verified because at this time the observed curve was complicated by the effects of emptying and refilling the tank. The 2- or 3-month period of primary consolidation is not unreasonable in view of the multiplicity of drainage layers in the clay. However, this possible explanation for the shape of the observed curve involves something of coincidence, and it may be preferable to consider the observations as purely empirical data.

Differential Settlements

After the tank was emptied, settlement reference points were established on the two nearby structures and readings were taken on them as well as on the tank. The increase in settlement from the time the tank was refilled until 1946 is shown in Fig. 1a. The settlements are plotted as the percentage of the maximum observed settlement, which occurred at the edge of the mat closest to the buildings. The mat remained unbroken and practically plane during the settlement. On account of the presence of the adjacent buildings the tank tilted slightly in their direction, but not enough to justify any alterations.

Figure 1b shows settlement contours computed on the basis of comparatively simple assumptions concerning the subsoil. It was assumed that the mass of clay could be divided into two principal layers, one of which extended from a depth of 18 ft to a depth of 32 ft and the other from 32 ft to 52 ft. The upper clay layer was assumed to have an average liquid limit of 44.3 per cent, and the lower layer a liquid limit of 38 per cent. The value of the compression index was determined by means of the statistical relationship

$$C_c = 0.009\ (L_w - 10\%)$$

in which L_w is the liquid limit expressed as a percentage of the dry weight. This equation has been found to give satisfactory results for normally loaded clays having a water content close to the liquid limit. The stresses were computed at midheight of each layer on the assumption that the subsoil was elastic, homogeneous, and isotropic, and the settlement was computed beneath each of the points at which settlement observations were made and at a few additional points.

The computed maximum settlement occurred near the middle of the tank and amounted to 1.04 ft, and the computed average settlement of the tank was approximately 0.87 ft. The computed settlement contours shown in Fig. 1b show very satisfactory agreement with the observed contours in Fig. 1a when it is remembered that the tank actually had a rigid base whereas a flexible base was assumed in the computations.

The computed average settlement of the tank, 0.87 ft, compares with the settlement of 1.1 ft actually observed in 1946. The actual settlment will doubtless continue to increase slowly. Therefore the approximate settlement computation indicated settlements somewhat smaller than those actually observed. Nevertheless, had the results been available before construction they would have given adequate warning of the probable behavior of the structure. If a settlement of 0.20 ft is considered to be due to the compaction of the loose sand above the clay, the observed settlement in 1946 due to the clay alone was of the order of 0.9 ft. On this basis the agreement between computed and observed settlements is quite satisfactory. In reality, a detailed discussion of the relation between the absolute values of the observed and computed settlements has no great significance because the observed time-settlement curve cannot be divided with certainty into two parts, one representing the primary consolidation and one representing secondary compression.

Conclusions

In this paper the history of a water-treating plant founded above a deep bed of stratified sand and clay has been described. A curve of observed settlement for the heaviest unit in the treating plant, the water tank, has been presented. It does not have the characteristic shape usually associated with time-settlement curves for structures on clay. It is probable, but not certain, that the deviation from the usual curve is due to the rate at which the load was applied.

The paper also indicates that a satisfactory conception of the settlement of the structure could have been obtained by means of very simple computations in which the compressibility is determined on the basis of its statistical relationship with the liquid limit. The liquid limit could have been ascertained by testing samples from a simple test boring such as that subsequently drilled to explore the soil conditions. The differential settlement between various parts of the treating plant could have been predicted very accurately.

IV. SETTLEMENT OF FOUNDATION DUE TO SATURATION OF LOESS SUBSOIL

O. K. Peck and R. B. Peck

Summary

This paper contains a description of settlements that occurred when the loess subsoil of a structure was unintentionally saturated.

Description of Structure

The structure under consideration is a one-story building, approximately 327 by 200 ft in plan. It was built in 1930 for servicing steam locomotives. As shown in Fig. 1, most of the floor area is occupied by 16 pits each about 4 ft wide, between 4 and 6 ft deep, and about 120

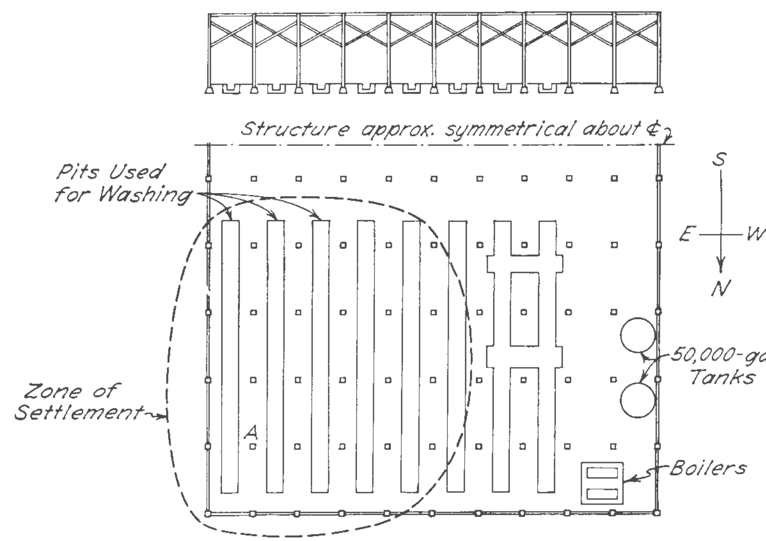


Fig. 1. Plan of Structure Whose Subsidence Due to Loess Subsoil Was Studied

ft long. The pits provide access to the under sides of the locomotive frames. The rest of the structure is occupied by storage space, a machine shop, a boiler, and two water tanks, each of 50,000-gal. capacity.

The framework of the structure is of precast concrete with a timber roof. The columns support little load except that of the roof. They rest upon small concrete pedestals that for the most part have

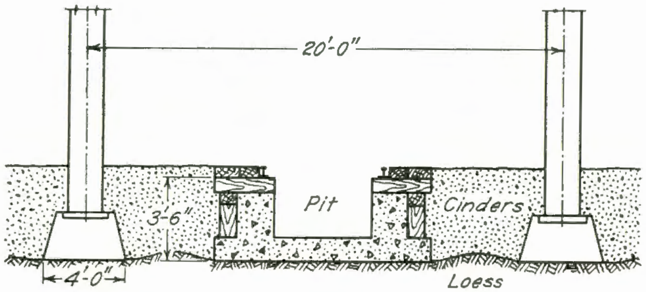


Fig. 2. Cross-Section Through One of the Engine Pits

bases 4 ft square. The maximum design load at the base of these pedestals was 1.6 tons per sq ft.

Figure 2 shows a cross-section through one of the engine pits. It consists of a concrete trough upon which rests timber blocking that supports the rails. On either side of the pit, the engine-house floor consists of cinders resting upon the original ground surface. The cinders were originally not covered with impervious material.

Foundation conditions appear to have been uniform over the entire site. The soil was originally described as a silt with sufficient cementing material to make it cohesive and very stable. It was necessary to use picks for all excavations for the footings. Several heavy structures had previously been erected in the vicinity. None of these structures had experienced sufficient settlement to raise any suspicions concerning the quality of the foundation material. Within the engine house itself no settlement was experienced in connection with the relatively heavy boilers or the two 50,000-gal. water tanks.

History of Settlement

Less than a year after construction of the engine house serious settlements were observed in the northeast corner. The six pits on the north end of the building had settled between 1 and 6 in. Several columns of the eastern portion of the house settled by amounts varying from 1 to 15 in.; the maximum value occurred at Column A, Fig. 1. The settlement caused a certain amount of structural damage to the north wall, which was pushed out of plumb by the action of cross bracing between the columns.

It was found that the northeast corner of the building had been used extensively for washing out locomotives, whereas the other parts of the structure were never used for this purpose. Where the washing

had been done, large amounts of water were spilled onto the area between the pits. Since these areas had no impervious surface, the water readily sank through the cinder fill and into the underlying soil upon which the footings rested. As soon as the subsoil became more or less saturated, very rapid subsidences occurred.

A closer examination of the subsoil indicated that the material was a gray-brown loess containing vertical root holes. The pronounced root-hole structure demonstrated that the loess had never before been saturated and that its original highly porous structure was intact at the time the building was constructed. Tests on small samples of the material indicated a natural porosity of 46 per cent. The effective grain size of the material was 0.013 mm and the uniformity coefficient 5.8. These values are typical of wind-blown loess. It is apparent that the large quantity of water was sufficient to dissolve the cementing substances and to permit the structure of the loess to collapse. Furthermore, since the engine pits cracked during subsidence, it is possible that a certain amount of the soil was subsequently washed into the pits and removed.

It was found necessary to reconstruct some of the footings and to reduce the design load to 0.5 ton per sq ft where the structure of the material had been completely destroyed. A surfacing of impervious rock asphalt was placed over the area where washing operations were carried out, and additional drainage facilities were provided. During the 15 years since these repairs were made only very minor subsidences have occurred.

Conclusions

The preceding example indicates that the structure of a very rigid undisturbed loess deposit even with a porosity as low as 46 per cent can be irreparably damaged by flooding. As a consequence, the surface of the deposit may settle and buildings founded on the deposit may subside and crack. The settlement appears to be caused almost exclusively by the destruction of the bond between the particles of the loess and to be practically independent of the intensity of the loading.

V. MEASUREMENTS OF PRESSURES AGAINST A DEEP SHAFT IN PLASTIC CLAY

R. B. PECK AND SIDNEY BERMAN

Summary

This paper describes the method of construction of a deep circular shaft in Chicago, the soil conditions adjacent to the structure, and measurements made to determine the lateral pressure exerted by the soil against the lining of the shaft.

Description of Excavation

Construction of a pump and sump for the Chicago Subway required the excavation of a circular shaft extending to a depth of 72 ft below the street surface. The top of the shaft was located 11 ft below the street surface, within the basement of an old six-story building. These conditions are shown in the Plan and Section, Fig. 1. It is seen that the space occupied by the shaft was not located under the building proper, but was in a vaulted space beneath the sidewalk.

For a depth of 30 ft below the basement floor, the shaft had a diameter of 15.5 ft. For the remainder of the depth the diameter was 12 ft. The lining of an existing shield-driven tunnel, 25 ft in diameter, was located 2.5 ft from the wall of the shaft at the nearest point. The center line of the tunnel was at a depth of 27.5 ft below the basement floor.

The soil from the street surface (El. +13.0 Chicago City Datum) to a depth about 2 ft below the basement floor was a miscellaneous fill of cohesionless material in the upper portion, and a gray silt in the lower. The underlying material for the entire depth of the shaft was a plastic glacial clay, for the most part quite soft. The unconfined compressive strength of the clay is shown in Fig. 1. The liquid and plastic limits were respectively 34 and 18 per cent. Above El. −22 the natural water content was approximately 27 per cent, between El. −22 and −44 the value was approximately 23 per cent, and below El. −44 about 18 per cent. Water level was about 1 ft above the clay surface.

The excavation was made with a clamshell bucket, and necessary hand trimming done with clay knives. After each advance in depth of 2 ft 9 in. a set of bracing was placed before further excavation.

The bracing consisted of 9-in. channels curved to a circular shape corresponding to the diameter of the shaft. Each circular ring was fabricated in three segments for ease in erection. At the ends of each segment a butt plate was provided, with holes for two bolts by means

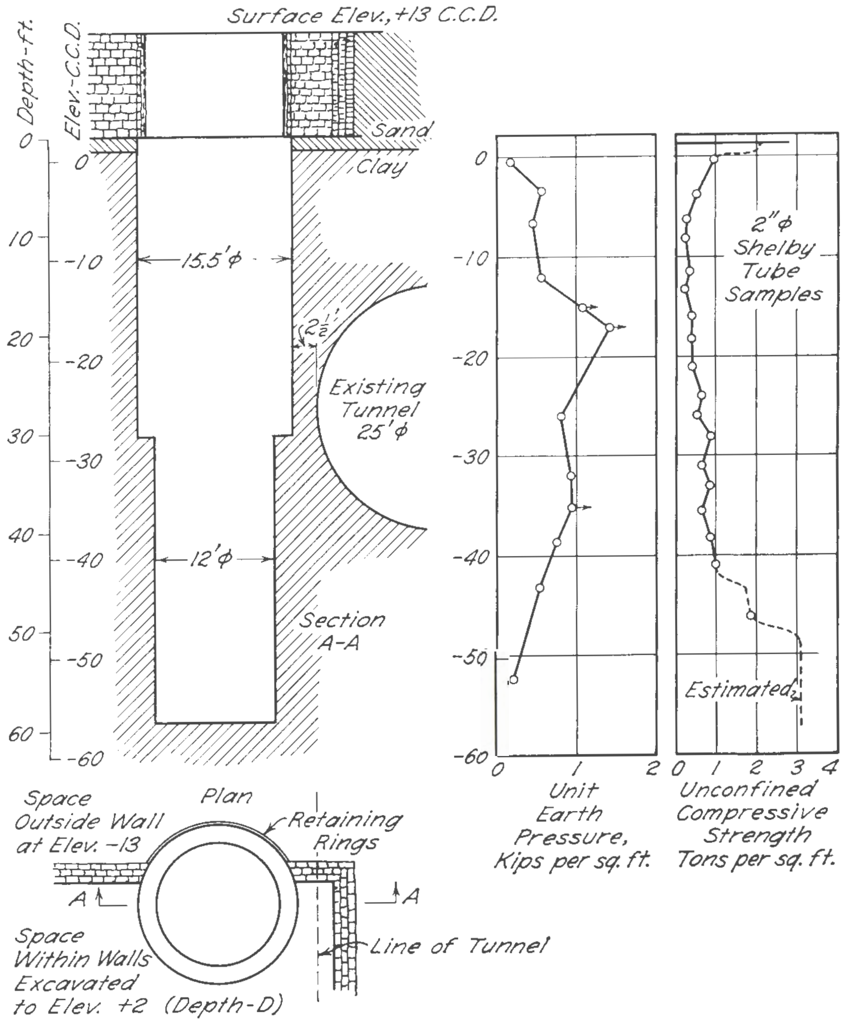


FIG. 1. PLAN AND SECTION OF CIRCULAR SHAFT IN PLASTIC CLAY

of which the three segments could be fastened into a complete ring. When excavation had advanced a distance of 2 ft 9 in. below the last previous ring, a new ring was assembled and suspended from the one above by angle spacer bars 2 ft long, as illustrated in Fig. 2. Steel lagging plates were placed one by one to span the distance between the lowest two rings. The plates were placed behind the rings so that the upper and lower 3 in. of each plate were supported by the

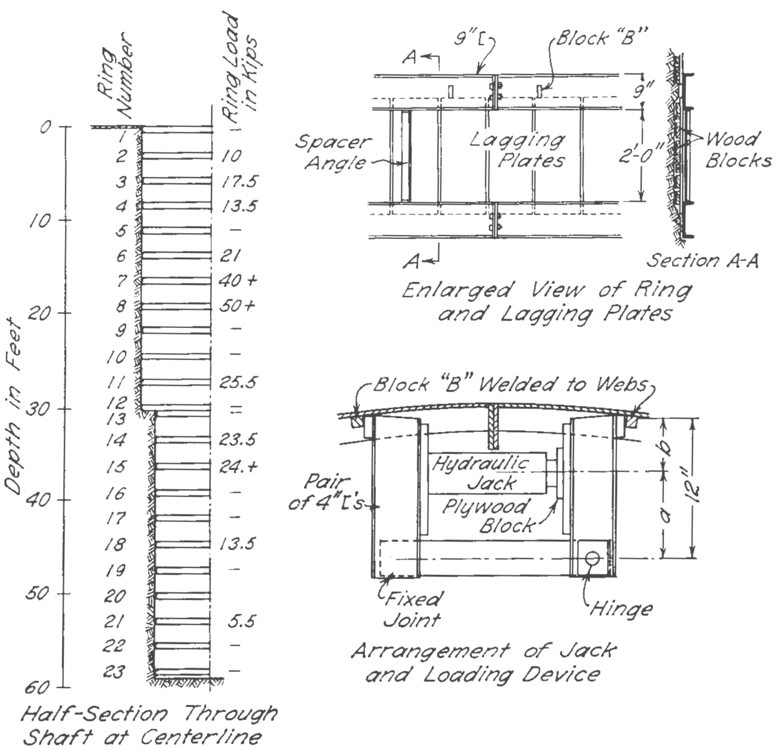


FIG. 2. ARRANGEMENT OF BRACING WITHIN CIRCULAR SHAFT

rings. After each plate was placed, small wood blocks, usually 10 in. long and 1 in. thick, were driven behind it until any voids due to clearance between lining and clay were filled. In this manner, complete bearing of the lining was assured for the entire periphery of the shaft. Excavation was done carefully so that voids more than 1 in. deep seldom existed even before blocking. No connections were required between the lagging plates and rings. Squeeze of the clay was probably reduced to a minimum, as evidenced by the fact that the maximum settlement of the adjacent building during excavation was ⅝ in. About three weeks were required to excavate and brace the shaft.

Measurement of Pressures

To ascertain the pressures exerted by the clay against the lining, it was most convenient to determine the direct compressive stress carried by each ring. The area of each ring was known to be far greater

than that required to support the probable load, but the rings were selected on the basis of availability rather than economy. Hence, the anticipated stresses were very small. This fact, together with the rough handling necessary in erection, made successful direct strain determinations extremely improbable. The use of a hydraulic jack equipped with a pressure gage appeared to hold greater promise and was adopted.

In general, the procedure for measuring the load in a ring was to weld a steel block 1 in. square and 4 in. long on each side of one of the joints in the ring, to loosen the bolts connecting the segments, and to determine the load by means of a hydraulic jack reacting against the blocks. The blocks (B) are shown in Fig. 2. The presence of the butt plates, however, prevented the insertion of the hydraulic jack directly between the blocks. To overcome this difficulty the loading device shown in Fig. 2 was constructed. It provided a lever system for transferring the jack load to the blocks from a point inside the butt plates. Measurement of the lever arms *a* and *b* provided the necessary information for computing the load at the blocks from the jack load.

The curvature of the rings caused the loading device to bear on the inner corner of each block B. In order to apply the load as near to the neutral axis of the channel as possible, shims were inserted which caused the load to be applied about ½ in. from the channel webs. It was observed that the slight eccentricity of the applied load did not cause the joint to open unevenly. In every case, the separation of the butt plates occurred uniformly.

Eleven of the 22 rings in the bracing were tested as soon as the shaft was completed. The measured loads are shown in Fig. 2, and corresponding unit pressures against the shaft in Fig. 1. For the three ring loads after which a + sign appears, the capacity of the equipment was exceeded. On rings 7 and 15, the welds failed to hold the blocks B to the channels. In these two cases the behavior of the apparatus indicated that the ring load was probably not greatly in excess of the recorded value. In ring 8, however, the load of 50,000 p.s.i. was sufficient to buckle the loading device without any indication that the ring load was closely approached. Ring 23 was embedded in the concrete floor slab when the measurements were made.

The pressure displayed a marked reduction in the lower part of the shaft, similar to the arching observed in open cuts upon which measurements were made. The magnitude of the pressure was, however, only about 46 per cent of that computed according to the method proposed for the open cuts (see Trans. ASCE, 1943, p. 1008).

Conclusions

On theoretical grounds it has been stated that the pressure relief due to the transfer of shearing forces to the soil beneath the bottom of a shaft in clay is likely to be very important (see Terzaghi, Theoretical Soil Mechanics, p. 214). The measurements described in this paper confirm this statement, and furnish empirical data concerning the magnitude of pressures against one such shaft.

VI. DESCRIPTION OF A FLOW SLIDE IN LOOSE SAND

R. B. Peck and W. V. Kaun

Summary

This paper describes a flow slide that occurred in fine loose sand during reconstruction of a dock wall in East Chicago, Indiana, in 1946. The flow occurred through a narrow opening made by the removal of a small portion of a sheet pile wall. Since no seepage pressures were involved, the primary cause of the occurrence appears to be the collapse of the extremely loose structure of the fine sand.

Description of Slide

Conditions prior to the slide are shown in Fig. 1, which represents a cross-section through the dock wall. The outer face of the dock consisted of sheet piles 45 ft long. To the east of the sheet-pile wall was a canal with its water level at El. 0.0. The channel was maintained by dredging to El. −23. On the west side of the sheet piling the surface of the ground had a constant level at El. +7. A row of anchor piles 30 ft long was driven vertically at a distance of 40 ft from the sheet piling and was tied to it by means of steel rods at water level.

The material between El. +7 and El. −12 consisted principally of loose sand deposited by means of a hydraulic dredge. Most of this fill was pumped out of the channel during the construction of the dock. Between El. −12 and El. −25 was a natural deposit of beach sand of the same grain size but of much greater relative density. At El. −25, the sand throughout this general area rests upon a bed of medium clay. At the location of the dock, however, part of the original surface of the clay had been once lowered by erosion and a fill of organic

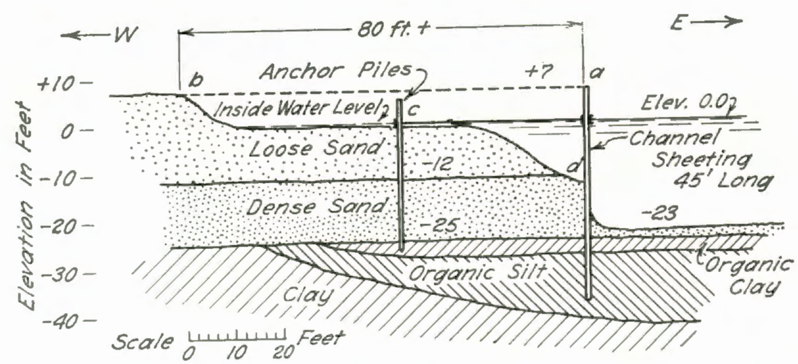


Fig. 1. Conditions Prior to the Slide: Cross-Section Through Dock Wall

material had accumulated before the beach sand was deposited. On account of the presence of the organic deposits as well as the inadequacy of the anchor piles, during the spring of 1945 the dock wall bulged toward the channel and showed signs of failure. As a consequence it was decided to reconstruct several hundred feet of the dock. In the reconstruction, longer piling was to be substituted for the original sheet piles and a new anchorage was to be built at a distance of about 80 ft west of the dock line.

In preparation for the repairs, all the material between El. +7 and El. 0.0 was removed to the west of the dock line for a distance of about 80 ft. In addition, a deeper pocket with a slope rising toward the west was excavated in the loose sand immediately behind the sheeting, so that soil pressure did not act against the sheeting above about El. −14. Upon completion of these operations, conditions were as shown in Fig. 1.

Inasmuch as the reconstruction operations were to extend to within a few feet of the south property line indicated by the fence in Fig. 2, the contractor took additional measures to protect the adjacent property from possible damage by sloughing of the slopes. At a distance of about 20 ft north of the fence, a row of sheet piles AB was driven at right angles to the dock wall. This temporary piling extended to approximately El. −27. Hence it barely penetrated the clay and organic material beneath the sand. A cross-section through the temporary sheet pile wall just west of the dock wall is also shown in Fig. 2. It indicates the manner in which the original bank near the property line was sloped from El. +7 to water level. At M, Fig. 2, was located a mooring post that also extended to approximately El. −27.

By virtue of these preliminary operations the contractor expected to be able to pull the sheet pile dock wall starting just north of point A with little difficulty and without the loss of much sand behind the dock line. However, upon removal of the first two piles, having a total width of 2½ ft, sand began to flow through the narrow opening and continued to flow for about 30 min before the gap could be closed. During this time between 500 and 800 cu yd of sand flowed through the gap.

The principal manifestation of the flow was the dropping of large chunks of sand into the water at the edge of the southern bank. These pieces first cracked away from the bank and then slumped into the water and disappeared. After the slide the outline of the bank was as shown in Fig. 2. Near the east end of the bank several cracks remained. The position of the ground surface just west of the bulkhead line is

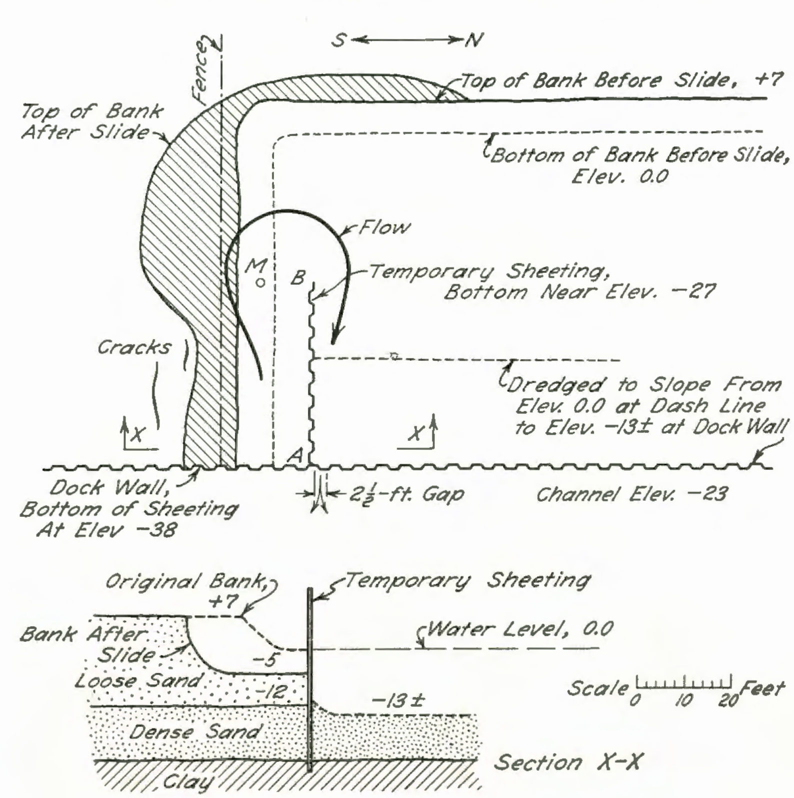


Fig. 2. Protective Measures, and Outline of Bank After Slide

shown in Section XX. In corner A the surface of the sand had descended from El. 0 to El. −5. At point B near the end of the temporary sheeting the depth was only slightly greater. At the place where the sheeting was removed, the surface of the sand was at about El. −11. Since the total distance from A to B around the end of the temporary sheeting and back to the gap was 80 ft, the final average slope was approximately 1 on 13.

The photograph, Fig. 3, shows conditions after the slide. At the rear of the view the fence can be seen in its original position. Closer to the dock, the fence has collapsed in the area involved in the slide. At the extreme right of the photograph can be seen the temporary sheet pile wall. It was still in position in spite of the fact that after the flow it had an embedment of only 12 to 15 ft. In the center of the photograph is the mooring post still in position and not tilted.

It is obvious from the drawings and the photograph that the slide was not a deep-seated phenomenon and that only the material above El. −12 was involved. Had the underlying sand layer been disturbed, the mooring post and the temporary sheeting would almost certainly have collapsed or moved. Inasmuch as the dock wall was provided with holes at water level to maintain the elevation of the water at the same level inside and outside the sheeting, no hydraulic head could have existed across the gap at the time the sheeting was removed. The relatively small disturbance caused by removing the support from a

Fig. 3. Conditions After the Slide (Photograph); Temporary Sheet Pile Wall and Mooring Post Still in Position

few feet of submerged sand at the gap set into motion a flow that continued even when an extremely flat slope was attained. The movement stopped only when the gap was closed.

The excessively loose nature of the overlying part of the sand deposit was indicated by a number of test borings made at the site before reconstruction operations were begun. Each of these borings was made within a 2½-in. casing. Samples were taken at intervals of 2½ ft in the vertical direction by means of a split spoon having an internal diameter of 1⅜ in. and an external diameter of 2 in. The spoon was driven into the ground by a drop hammer weighing 140 lb and allowed to fall 30 in. The number of blows per foot of penetration was recorded as a measure of the relative density of the sand. This procedure is widely used in the United States for preliminary exploration of sand

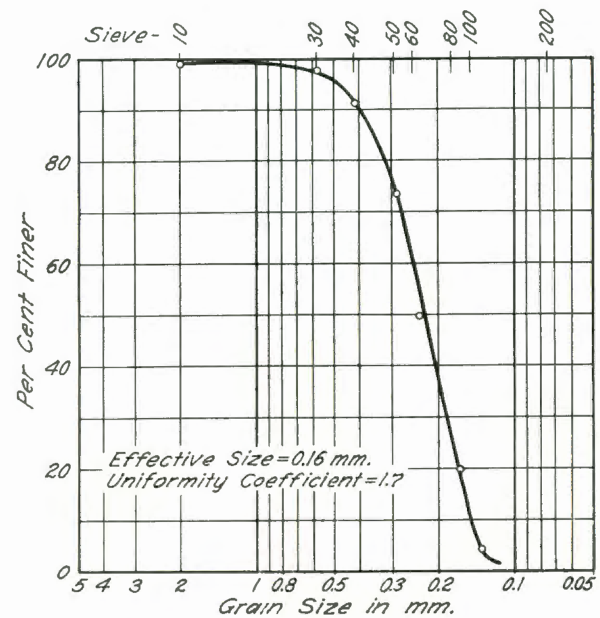


Fig. 4. Grain-Size Curve of Sample of One of Finer Lenses in Hydraulically Deposited Soil

deposits. In general, resistances of less than 5 blows correspond to very loose sand, from 5 to 10 to loose sand, from 10 to 30 to sand of medium density, and over 30 to dense sand.

Within the upper 12 ft a single blow of the drop hammer was sufficient to advance the sampler as much as 22 in. In many parts of the deposit the resistance to penetration was only one blow per foot. On the other hand, below El. −12 the penetration resistance varied between 10 and 15 blows. Hence, the lower sand appears to have been relatively loose but stable whereas the upper sand involved in the flow was abnormally loose and unstable. The grain-size curve of a sample of one of the finer lenses in the hydraulically deposited soil is shown in Fig. 4. It indicates that the individual lenses are extremely uniform. The fraction passing the 100-mesh sieve and retained on the 200 and that passing the 200 were examined under the microscope. Both consisted almost exclusively of angular quartz grains; very few rounded or sub-rounded grains were present. Grains passing the 200-mesh sieve were slightly more angular than those in the coarser fraction.

Conclusions

This paper describes the general features of a flow slide that occurred in very loose sand due to a relatively minor disturbance. The flow was confined exclusively to a uniform excessively loose hydraulic deposit having a penetration resistance, according to the procedure described in the text, of as little as one blow for 22 in. On the other hand, the underlying sand which had a relative density somewhat greater (about 10 blows per ft) did not participate in the movement. Inasmuch as no seepage pressures were involved, it may be concluded that the flow was due to the spontaneous liquefaction of the excessively loose sand.

VII. A STUDY OF SUBGRADE CONDITIONS ON A RAILROAD IN THE WESTERN UNITED STATES

T. H. Thornburn

Introduction

This paper describes various subgrade conditions which were observed during a field investigation of railroad roadbeds. The subgrade soils are classified and drainage facilities are described. A tentative explanation of the mechanics of soft-spot formation is presented.

For some time the major railroads of the United States have been interested in the origin, nature, and correction of subgrade instability. This problem became particularly acute during the war years, when the railroads were operating at capacity and were forced to perform roadbed maintenance with a minimum amount of labor. Unstable roadbeds resulted in the loss of thousands of dollars by necessitating "slow orders," abnormal resurfacing operations, and extra maintenance. Various railroads attempted to find cures for their instability problems. One of the most promising of these was pressure cement grouting. However, little work was done to establish the actual cause of the loss of subgrade support.

Qualitative observations have established that free water, medium to highly plastic subgrade soils, and heavy traffic all contribute to the development of "soft spots." In this respect the problem is similar to the "pumping" problem on concrete pavements encountered by our state highway departments. The question whether unstable track is primarily the result of poor subgrade drainage has long been a subject for discussion. In but few cases have investigators been able to determine that benefits from subgrade drainage lasted more than a year or two. Also, the relative importance of surface and subgrade drainage has never been satisfactorily established. Nor has enough information been collected to determine the amount or thickness of plastic clay required to start the development of soft spots.

To study these problems the Association of American Railroads and the Engineering Experiment Station of the University of Illinois entered into a cooperative agreement for an investigation to be carried on under the direction of G. M. Magee, Research Engineer of the A. A. R., and Dr. R. B. Peck, Research Professor of Soil Mechanics.

During the summer of 1946 it was observed that a portion of the Denver & Rio Grande Western Railroad about 40 miles south of Denver, Colorado, lay in an area which was especially suited to the type of investigation which was desirable. Within a distance of several hundred feet very unstable lengths of track alternated with stable lengths. Here, it seemed, was an excellent opportunity to arrive at some rather definite conclusions with regard to the effect of drainage and the influence of the character of the clay on soft-spot formation.

Geological Character of the Area

In the vicinity of Larkspur, Colorado, the railroad runs along the east side of a stream valley at the base of the foothills of the Front Range. East of the right-of-way the ground rises rapidly to a low range of hills which are part of the poorly developed and badly eroded western tableland. West of the right-of-way the ground slopes downward to the stream about 25 to 30 ft below the track. The track lies on alluvial material which is predominantly sand and gravel with lenses or strata of clay. This material appears to have been brought down from the mountains and subsequently eroded by the valley stream. Several small streams are crossed by the track in the area selected for investigation and these serve as surface drainage outlets for the track structure. Most of these streams are dry during much of the year, but are usually filled to the flood stage during the storms which occur in this vicinity.

General Subgrade Conditions

The investigation showed that the subgrade consisted of 8 to 12 in. of medium to highly plastic clay underlain by 5 to 12 in. of sandy clay which graded into a mixture, sometimes stratified, of sandy clay, clayey sand, and sand. At a depth of 60 in. below base of rail either gravel, sand, or clayey sand was generally encountered. Ordinarily the thickness of the upper clay did not exceed 12 in. and in no case was any free water found in the material below the clay layer. This was to be expected, since any permeable strata within 15 to 20 ft of the surface must have been readily drained by the numerous deep tributaries crossing beneath the track.

Field Observations of Trench Sections

Ten trench sections were studied during the course of this investigation. Six were opened during the summer of 1946; the remaining four, together with several auger borings, were studied in July, 1947. Since the nature of the subsoil and the drainage conditions were the primary concern, it was not deemed necessary to excavate directly beneath the track structure. Consequently the trenches extended only from the ends of the ties out to the ditch lines.

Figure 1, a diagram of each of the trench sections, indicates the

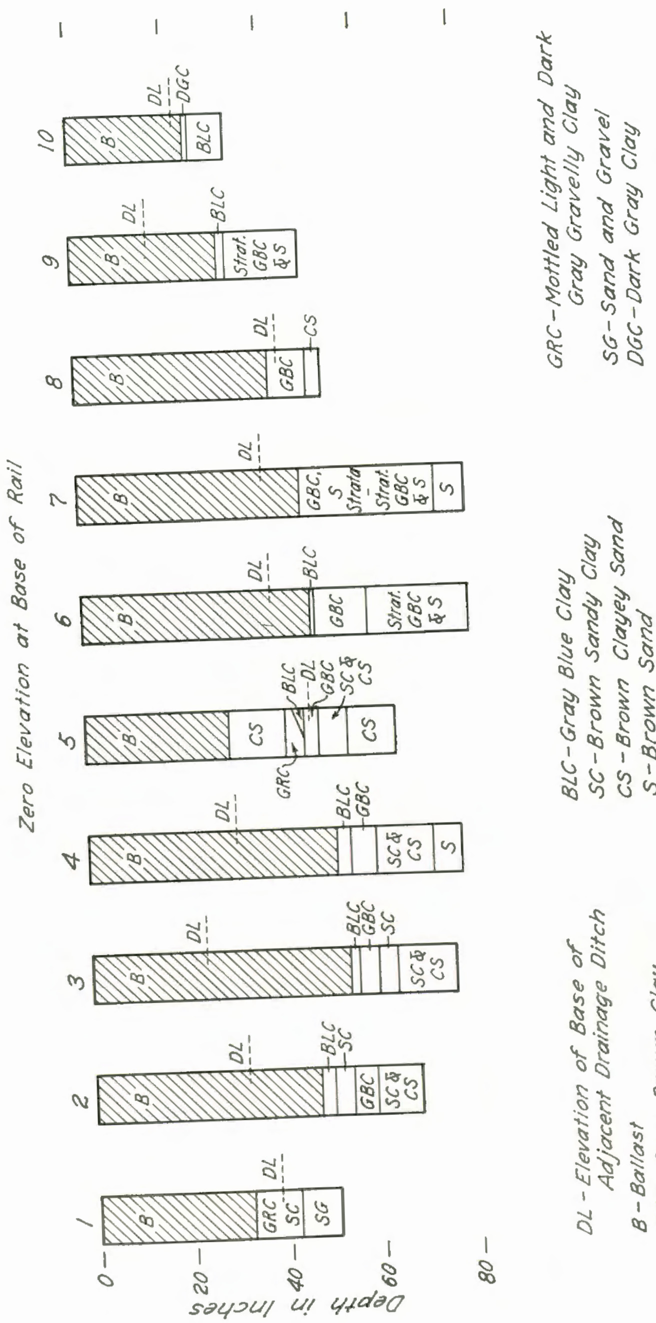


Fig. 1. Diagram of Trench Sections Showing Materials at End of Ties and Position of Ditch Line

materials which were encountered at the end of the ties, and also the position of the ditch line with respect to the track structure. Table 1 contains the index properties of the plastic soils which were found. Table 2, page 48, indicates the condition of the roadbed as classified by the track foreman, and the position of the ditch line with respect to the top of the subgrade.

Trenches 1 through 5 cover a total distance of only five rail lengths. Trenches 1 and 5 were opened at each end of a very troublesome stretch of track which had required maintenance as often as once a week in the past. Trenches 2, 3, and 4 were located within this unstable section just one rail length apart. Several years before the investigation a longitudinal French drain filled with large pieces of slag had been installed parallel to the track and about at the end of the ties. After the installation of the drain severe trouble was experienced about twice a year. Investigation disclosed that the drain had become clogged with fine material and had been displaced laterally by the squeezing of ballast and clay from beneath the track. Figure 1 shows that the surface drainage ditch could not function to remove the water from the deep pockets of ballast that must have been present below the central part of the roadbed. It was also apparent that the 8- to 12-in. layer of plastic clay was not permeable enough to transmit the accumulated surface water to the porous sand strata below, as free water was found in all the ballast pockets even in midsummer.

Trenches 6, 7, and 8 were opened in three different soft spots located about five rail lengths apart. Numbers 6 and 7 were both located in shallow cut sections and No. 8 was in a low fill section. All three were classified by the track foreman as very troublesome and required maintenance from 1 to 4 times a month. At all three locations attempts had been made to drain the ballast pockets by means of rock drains running out to the ditch line. In Nos. 6 and 7 this treatment was not effective, since the base of the ballast pocket was about 8 in. below the level of the ditch. The excavation of trench 8 disclosed that the track had been founded on a mass of plastic clay removed from the adjacent cut. Here the rock drains had been effective in removing the water from the ballast to the ditch, which was at about the same elevation as the subgrade but which was cut in the very permeable clayey sand topsoil of the area.

Trench 9 was opened at a cut-fill transition section. This location had formerly been very troublesome, requiring twice-monthly maintenance. In 1943, the track foreman had used a 5-in.-diameter post-hole auger to bore seven holes beside the rails through the clay subgrade down to clean white sand at a depth of 19 ft below the base

Table 1

Properties of the Plastic Subgrade Soils

Trench Number	Mottled Gray Gravelly Clay			Gray-Blue Clay			Gray-Brown Clay			Brown Sandy Clay			Brown Clayey Sand		
	L_w	P_w	I_w	L_w	P_w	I_w	L_w	P_w	I_w	L_w	P_w	I_w	L_w	P_w	I_w
1	38.2	16.6	21.2	...	...	...	...	...	...	47.5	19.0	28.5	...	...	...
2	...	...	...	64.2	25.9	38.3	61.3	28.2	33.1	35.1	15.4	19.7	...	...	...
	...	...	...	...	...	...	...	...	...	31.3	15.1	16.2	...	...	...
3	...	...	...	71.0	27.2	43.8	73.1	28.1	45.0	42.7	18.3	24.4	26.6	16.7	9.9
4	...	...	...	54.1	27.9	26.2	45.1	24.1	21.0	26.9	13.9	13.0	...	...	...
5	32.1	14.9	17.2	62.3	23.6	38.7	53.1	20.1	33.0	29.9	14.2	15.7	...	...	...
6	...	...	...	68.4	27.2	41.2	58.8	26.5	32.3	...	...	...	...	...	...
7	...	...	...	...	...	...	69.5	31.3	38.2	43.0	20.1	22.9	...	...	...
	...	...	...	...	...	...	55.0	24.2	30.8	...	...	...	...	...	...
	...	...	...	...	...	...	62.0	24.8	37.2	...	...	...	...	...	...
8	...	...	...	...	...	...	71.0	31.2	39.8	...	...	...	18.7	14.7	4.0
9	...	...	...	60.8	25.0	35.8	...	...	...	...	...	...	...	...	...
	...	...	...	68.1	27.2	40.9	...	...	...	...	...	...	...	...	...
10	34.6*	14.4	20.2	53.5	17.9	35.6	...	...	...	...	...	...	...	...	...

* Dark Gray Clay.

TABLE 2
CONDITION OF ROADBED AND POSITION OF DITCH LINE

Trench Number	Roadbed Condition	Position of Ditch Line Referred to Subgrade Surface
1	Stable	6 in. below
2	Unstable	15 in. above
3	Unstable	30 in. above
4	Unstable	21 in. above
5	Stable	5 in. below
6	Unstable	8 in. above
7	Unstable	8 in. above
8	Unstable	2 in. below
9	Unstable	15 in. above
10	Unstable	2 in. above

of rail. Since then, the track has been raised only about eight times. The foreman's records showed a ballast pocket formation extending to a depth of 4 ft directly below the rails. Apparently the auger holes are still functioning to remove water from the ballast, because the ditch line is well above the top of the subgrade.

Trench 10 was opened in an unstable low fill section. The trench could not be completed because of inclement weather, but the presence of a highly plastic clay subgrade was established. The instability began with one rail length in 1940 and by July, 1947, a length of four rails had become involved. There was evidence of the beginning of a ballast pocket accompanied by the squeezing of clay from beneath the track. It is probable that deepening of the adjacent ditch together with the installation of a few rock drains would readily correct the instability at this stage.

Discussion of Clay Soils Encountered

Figure 2 shows the position of points representing test results for each of the clay soils, on a plasticity chart of the type developed by A. Casagrande. With the exception of the location at trench 10, the uppermost stratum of the subsoil in the unstable sections was found to be a gray-blue or gray-brown clay of high plasticity. There is very little difference between the two different-colored clays except that the gray-blue clays exhibit plastic properties over a slightly wider

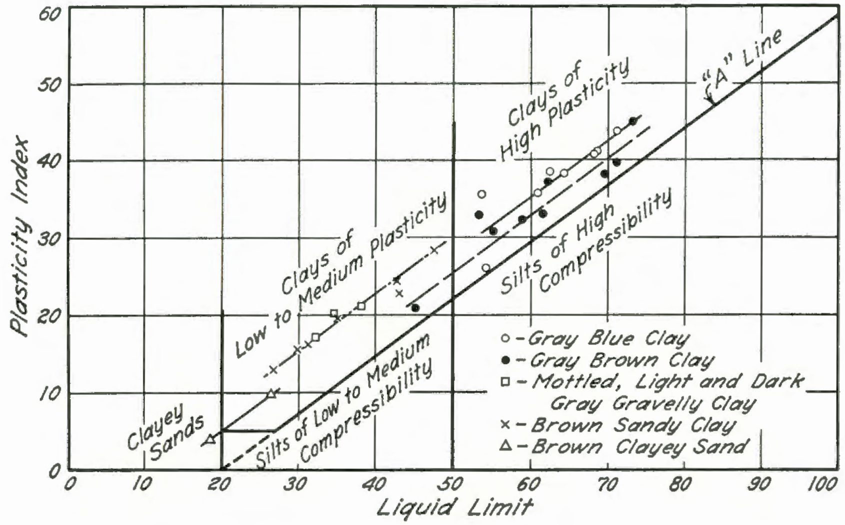


FIG. 2. POSITION OF POINTS REPRESENTING TEST RESULTS FOR EACH OF THE CLAY SOILS

range of moisture content than do the gray-brown clays. The gray-blue clays are probably less permeable and more cohesive than the gray-brown ones.

In the trenches in the stable sections, the uppermost stratum of the subgrade was found to be either a mottled light-and-dark-gray gravelly clay or a brown sandy clay. The plasticity chart indicates these soils to be clays of low to medium plasticity. These clays plot well above the A-line, which indicates a wide plastic range for clays of this type. It seems likely that the colloidal portion of these soils is similar to that of the gray-blue clays, but that the degree of plasticity varies with the amount of colloidal material present.

The inadequate examination of the soil profile in trench 10 indicated the uppermost soil stratum to be a dark gray clay similar to the upper clays in the stable sections. However, there was only about 1 in. of this material and it was underlain by several inches of highly plastic gray-blue clay.

In general the subgrade consisted of gray-blue clay underlain by gray-brown clay which in turn was underlain by brown sandy clay or stratified brown clay and sand. In some trenches the gray-brown clay was absent; in others, the gray-blue clay; but in all trenches except the incomplete No. 10 a stratum of brown sandy clay, clayey sand, or stratified clay and sand was found in the lower portion of the trench. Since these clays were usually stratified with sand they could be characterized as well drained. In no case was there any evidence of saturation of the sand strata associated with these clays. It is not likely that such clays would ever be the source of roadbed instability.

Discussion of Surface Drainage Facilities

Examination of the trench sections established the presence of highly plastic clay soils at or near the subgrade surface in every unstable location. However, the amount of water necessary to develop instability at the surface of the plastic subgrade cannot be determined easily. An indication that a considerable quantity of free water must be present before instability becomes a major problem is given by the data in Table 2. At every unstable location except that of trench 8 the ditch line was above the top of the subgrade rather than below. This indicates that any surface water collected by the ballast could not be removed by the ditch under any circumstances. At these locations free water is available to the subgrade throughout the year with the possible exception of the very driest season.

At the location of trench 8 the comparison of elevations is somewhat misleading, since the elevation of the top of the subgrade under the center of the track must have been below the clay surface at the end of the ties. This conclusion was verified by the flow of water from the ballast when the rock drains were installed by the track foreman. The improvement in stability of the roadbed brought about by such a simple device as the installation of rock drains indicates that free water must be available to the plastic clay in considerable quantity before maintenance becomes a serious problem.

Mechanics of Soft Spot Formation

A study of the evidence provided by this field investigation suggests the following explanation of the formation of "soft spots" and ballast pockets in railroad roadbeds. Undoubtedly there are some sources of instability which are not adequately explained. However, the explanation is intended to cover the majority of cases and not the exceptions. It assumes that when the roadbed was established all fills were placed with a reasonable amount of care, so that large pore spaces which could hold large amounts of water within the fill itself were not present. It also assumes that the roadbed was provided with adjacent ditches, through cut sections and at grade, which did function to drain the surface of the subgrade for a time. It is intended to explain how instability can develop even though drainage may be provided by means of adjacent ditches or by perforated pipe or tile within the subgrade itself.

a. Where medium to highly plastic clay is present at or near the top of the subgrade, the weight of the track structure, including ballast, plus present-day locomotive wheel loads is sufficient to cause consolidation of the clay unless it has been thoroughly compacted. (Few fills and practically no cuts were subjected to controlled compaction prior to 1940.)

b. A relatively slight amount of settlement beneath the track produces a depression capable of holding free water transmitted to the subgrade through the pervious ballast. Even though the subgrade may have been provided with adequate lateral drainage, the adjacent ditches can no longer function to remove all of the free water from the subgrade.

c. In the presence of free water the surface layer of the highly plastic clay is dispersed by the action of traffic, and a lubricated surface is produced along which the lower part of the ballast may move outward toward the side of the track.

d. At the same time the presence of the excess water and the kneading action of the ballast under traffic cause the clay to soften to a depth of several inches and to raise its moisture content well into the plastic range. This permits plastic deformation of the subgrade itself, and the clay is also squeezed to the side of the track.

e. The condition of instability then becomes self-aggravating, since a berm of impermeable material is thrown up at the sides of the track whereas the subgrade surface under the track is depressed. The ballast pocket formation is then well developed and is capable of retaining large amounts of water, which are present even in the driest season if the subgrade is sufficiently impermeable.

f. At this time the roadbed apparently can be stabilized by some simple method of drainage which will remove the free water from the subgrade ballast pocket. This appears to be possible, since the track has been consolidated under a given set of loading conditions and equilibrium is established or may soon be established if the clay subgrade has no access to free water. However, an increase in traffic loads may upset the equilibrium, causing further consolidation and again rendering the drainage installation useless. Any other condition, such as clogging of the drains, which again permits accumulation of free water will allow the conditions of instability to return.

Conclusions

A field investigation of stable and unstable sections of railroad track on the Denver & Rio Grande Western Railroad near Larkspur, Colorado, has yielded the following information.

a. Unstable track is associated with moderate to highly plastic clay soils at or very near the surface of the subgrade.

b. A highly plastic clay layer as thin as 1 in. located at the subgrade surface apparently may cause lateral displacement of the ballast when a sufficient quantity of free water is present.

c. Sand strata as close as 12 in. to the surface of the subgrade are not effective in draining the ballast, if covered by highly plastic clay.

d. Considerable quantities of free water appear to be necessary to promote serious instability since pervious drains to the ditch line or to the sand below the clay have been effective in increasing the stability of very unstable sections of track.

e. Moderately plastic clay subgrades did not become unstable where surface water was effectively removed by adjacent ditches.

A tentative explanation of the mechanics of soft spot formation in railroad roadbeds has been presented.

VIII. EFFECT OF NATURAL HARDENING ON THE UNCONFINED COMPRESSIVE STRENGTH OF REMOLDED CLAYS

Oreste Moretto

Purpose

In the literature of soil mechanics, much discussion has centered on the source of the strength and rigidity of undisturbed clays. Until recently, this question appeared to have only academic importance. The results of field observations carried out during the last few years, however, have suggested that the question may have very important practical implications in connection with the consolidation of hydraulic fill dams, the remolding due to driving of piles, etc.

The great decrease in strength experienced by some clays when remolded at unaltered water content was first observed by the Swedish Geotechnical Commission during the investigation of landslides. Subsequently, A. Casagrande[1] observed that the consolidation characteristics of undisturbed and remolded clays were also markedly different, and he concluded that such disturbing events as the driving of piles into soft clay are likely to increase the compressibility of the clay to such an extent that the piles may actually be detrimental.

In order to explain the difference in the physical properties of undisturbed and remolded clays, Casagrande proposed a theory according to which the clay particles settle during the process of sedimentation into a definite arrangement called the "clay structure." His conception of the structure is that of a relatively coarse-grained skeleton of silt cemented together by highly compressed clay. The interstices of the skeleton are filled with soft clay. Casagrande states that "the building up of such a structure is chiefly dependent on the exceedingly slow process of natural sedimentation and consolidation" because a rapid increase of pressure during sedimentation would displace the grains before they could become bonded by highly consolidated clay. Remolding is presumed to destroy the connecting links between the larger soil grains and to replace them by the unconsolidated soft clay that fills the interstices. Since the development of the links is presumably dependent on the exceedingly slow process of sedimentation, Casagrande was led to the conclusion that "if we destroy the structure which nature has taken many centuries to build up, we cannot restore it."

[1] The Structure of Clay and Its Importance in Foundation Engineering, Journal of the Boston Society of Civil Engineers, April, 1932.

A fundamentally different explanation of the manner in which undisturbed clays acquire their strength and rigidity has been given by K. Terzaghi.[2] According to this theory, the strength and rigidity are acquired primarily by "slow physico-chemical processes" which are due to the surface activity of the mineral grains. As a consequence of its surface activity, each clay particle is surrounded by a shell of adsorbed water, almost solid near the particle, and quite viscous within a somewhat greater distance. During sedimentation, the mass of clay consolidates and the viscous layers merge. Upon further consolidation, the solid parts of the water shells may come into contact and merge at a number of points in the clay mass and, as a consequence, the mass becomes stiff. Remolding breaks the contacts between the solid water shells, displaces the grains, and introduces viscous adsorbed water between them, whereupon the clay becomes plastic.

Field evidence has been advanced in support of both conceptions.[3] However, since the field evidence has not been considered conclusive, laboratory experiments were designed to furnish pertinent data on the subject with the aim of throwing light on this much discussed problem.

Materials and Properties

The results of tests on four types of clay, identified by their places of origin, are reported in this paper.

Table 1 gives the physical properties of the clays. The natural sensitivity indicated in column 4 denotes the ratio between the unconfined compressive strength of the undisturbed material and the unconfined compressive strength after remolding at constant water content.

Figure 1 gives the grain-size distribution and the mineralogical composition of these clays, plotted as a function of the size of the particles. The mineralogical composition of the coarser fractions was determined microscopically and of the finer fractions by means of the differential thermal curves for the specimens.

Figure 2 shows the stress-strain relations for unconfined compression tests of the clays in the undisturbed and remolded conditions at the same water content. The unconfined compressive strength of the remolded samples, and of those undisturbed specimens that failed by bulging, was considered to be the stress corresponding to 10 per cent strain.

[2] Undisturbed Clay Samples and Undisturbed Clays, Journal of the Boston Society of Civil Engineers, July, 1941.

[3] See discussion of "Application of Soil Mechanics in Designing Building Foundations" by A. Casagrande and R. E. Fadum, Trans. A.S.C.E., 1944.

TABLE 1
PHYSICAL PROPERTIES OF CLAYS

Type of Clay (1)	Specific Gravity of Solid Matter (2)	Natural Water Content % (3)	Natural Sensitivity (4)	Liquid Limit % (5)	Plasticity Index % (6)	Origin of Clay Sample (7)
Laurentian	2.67	85±	14.0±	66.0	41.0	Beauharnois, Quebec, Canada; 6 to 9 ft below ground
Detroit I	2.73	24±	2.5±	29.2	12.7	Detroit, U.S.A.; 40 to 70 ft below ground
Detroit II	2.77	47±	6.0±	51.2	26.4	Detroit, U.S.A.; 25 to 40 ft below ground
Mexico	2.69	54±	5.0±	60.8	20.4	Mexican Dam; 40 ft below crest of dam

Test Procedure

The investigation consisted of determining the unconfined compressive strength of the clays when, after complete remolding, the material had been allowed to rest at constant water content for different periods of time. Tests of this kind were made for various water contents or relative consistencies of the materials. The relative consistency is defined as the difference between the water content and the plastic limit divided by the plasticity index.

For each type of clay except the Laurentian clay, enough material was mixed together to furnish a uniform mass sufficient for all the series of tests performed on it. This procedure could not be followed in the case of the Laurentian clay, and as a consequence the material was slightly different for the different series. However, the difference in the properties of the various Laurentian clay samples was not great enough to influence the results significantly, and the results can be compared with each other.

From each clay compression specimens were prepared in prismatic molds, Fig. 3, consisting of a bottom plate of glass, two L-shaped side plates of brass, and a top plate of glass. In the first tests the plates were lined with ordinary waxed paper to avoid sticking of the clay to the mold. Later, a thin coat of mineral vaseline was used instead of the waxed paper.

One of the specimens so made was tested immediately (0-day test). The rest were sealed in their molds by welding the brass and glass plates together with a fillet of paraffin, and the entire assembly was placed in a tin can containing paraffin on the verge of solidifying. The tin can was later filled and sealed with paraffin.

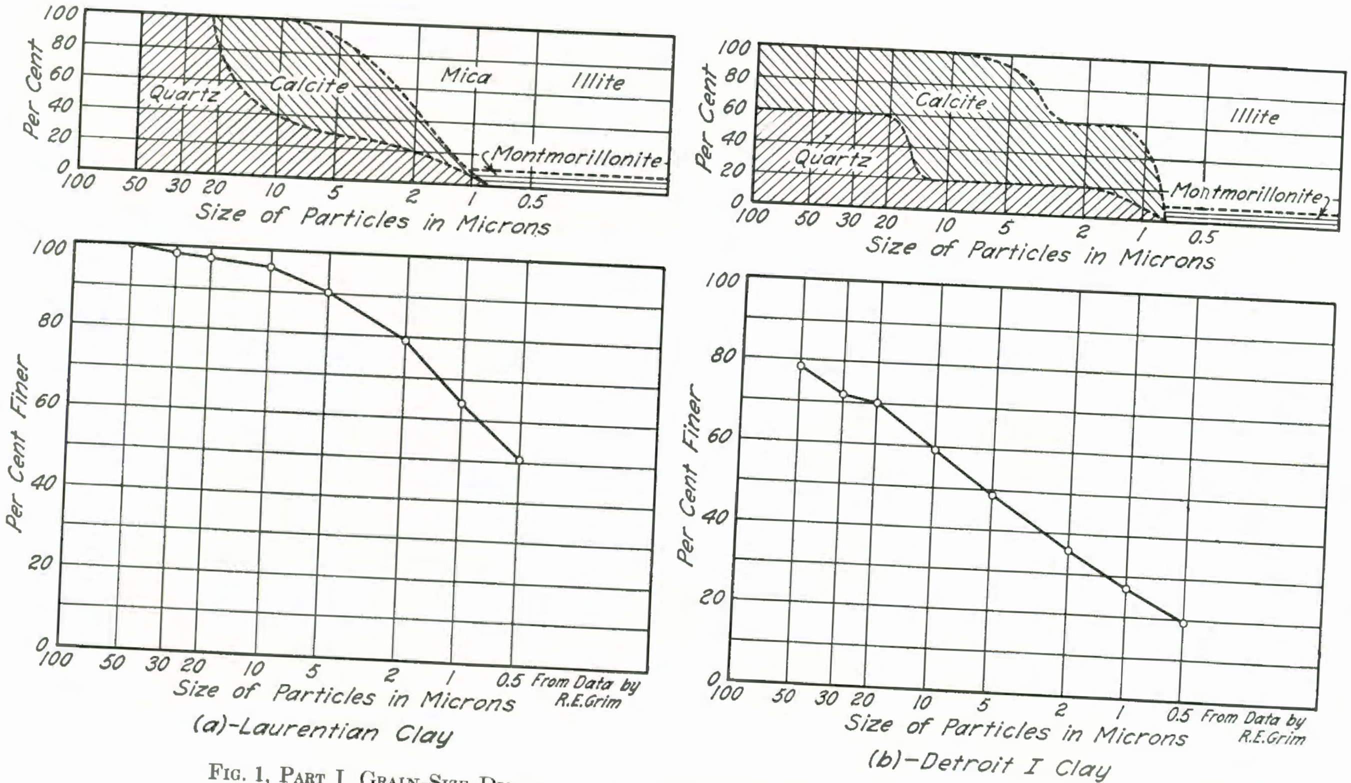


Fig. 1, Part I. Grain-Size Distribution and Mineralogical Composition of the Clays

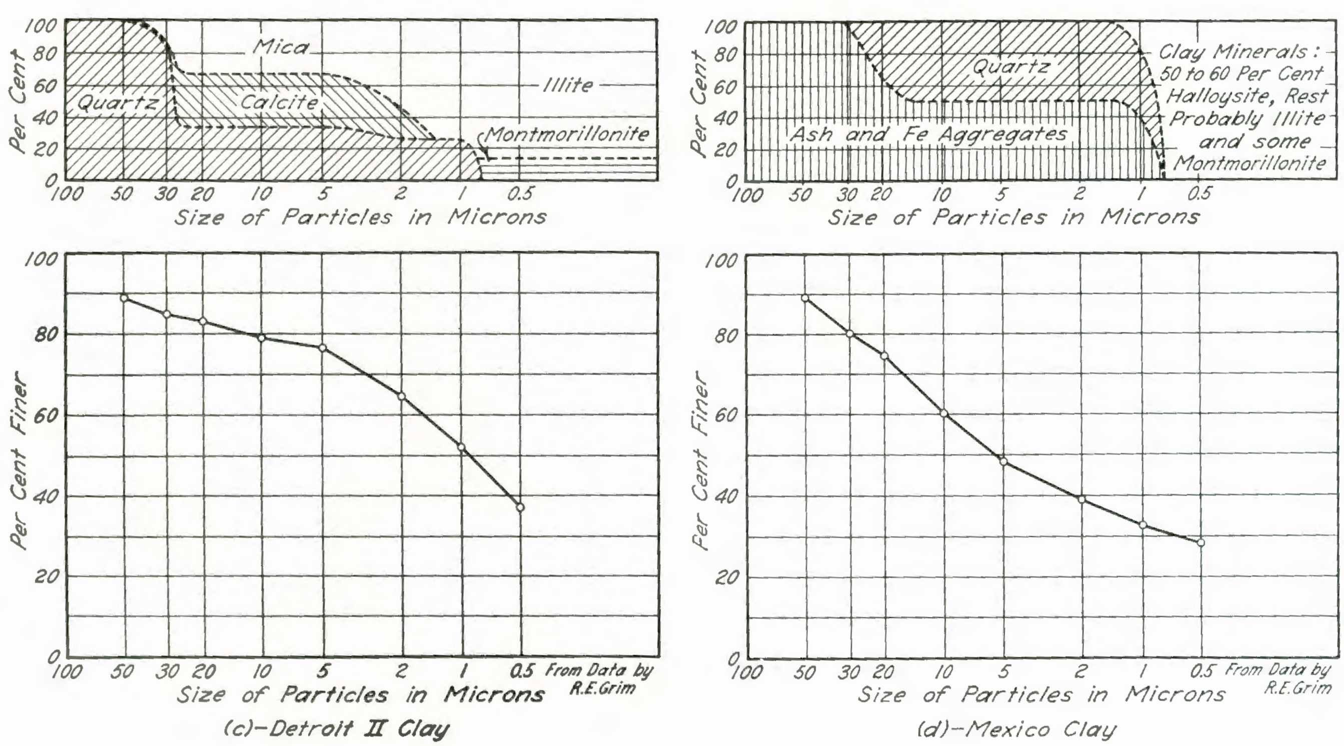


Fig. 1, Part II. Grain-Size Distribution and Mineralogical Composition of the Clays

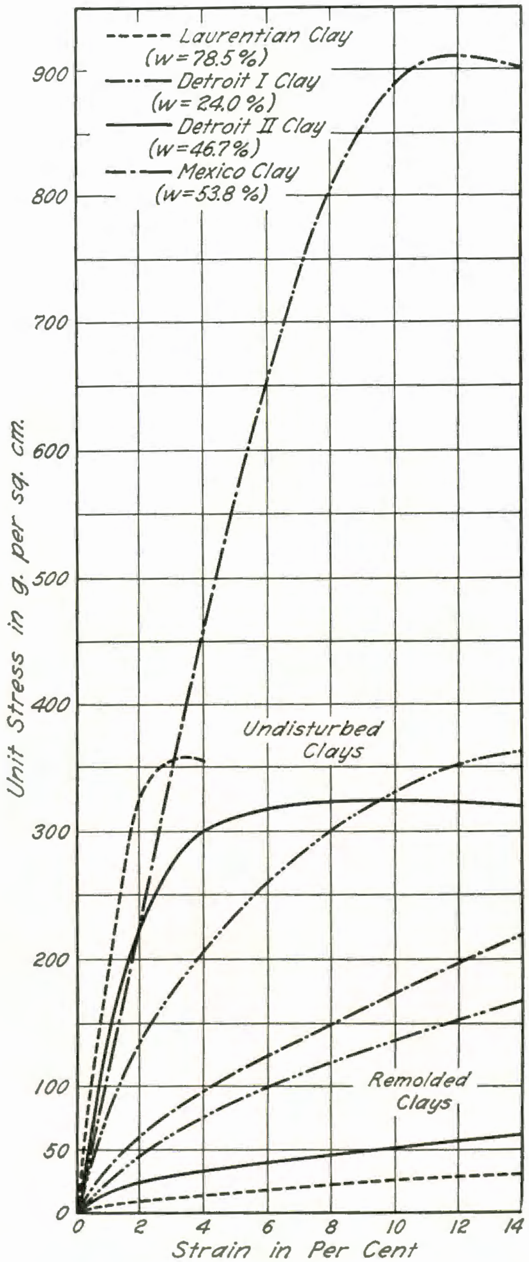


Fig. 2. Stress-Strain Relations for Unconfined Compression Tests

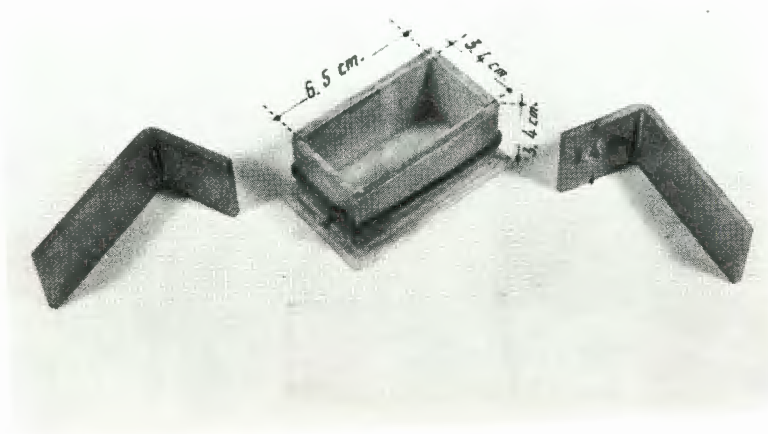


Fig. 3. Prismatic Molds for Specimens

The specimens, stored in this manner to keep them at constant water content, were tested after periods of 3, 7, 14, 28, 60, etc., days to determine the increase in strength they had experienced with respect to the 0-day specimen.

In general, the test procedure gave good results. However, the tests indicate a slight trend of the moisture content of the clay to decrease with age, mainly on account of absorption of water by the waxed paper. This decrease was small and did not significantly influence the results.

Results

The test results are given in Tables 2–5 and Figs. 6–15. The acquired sensitivity is the ratio between the ultimate strength of the clay after a certain period of rest and the ultimate strength immediately after the specimens were molded (0-day test).

a) *Laurentian clay.* — The experiments on Laurentian clay comprise 7 series in each of which the material had a different water content. Series 1 constitutes the main series. The results are given in Table 2, Fig. 4, and Fig. 5. Figure 4, showing the stress-strain curves, indicates that the clay experienced a very large increase of both strength and rigidity as a consequence of resting at constant water content. The sensitivity after 610 days of rest reached a value of 4.45, nearly one third of the natural sensitivity of the undisturbed material as given in Table 1.

TABLE 2
UNCONFINED COMPRESSION TESTS ON LAURENTIAN CLAY: SERIES 1
(Relative Consistency = 0.99)

Tested After a Rest Period of (Days) (1)	Water Content, % (2)	Ultimate Strength (for 10% Strain or Less) gm/cm² (3)	Type of Failure (4)	Acquired Sensitivity (5)
0	66.7	25.7	bulge	1.0
3	66.3	49.4	shear and splitting	1.92
7	66.0	51.0	shear and splitting	1.96
14	66.0	56.2	shear and splitting	2.19
28	65.5	63.0	shear and splitting	2.45
60	65.0	83.0	shear and splitting	3.22
120	65.3	90.1	shear and splitting	3.51
240	65.5	109.0	shear and splitting	4.25
610	65.6	114.0	shear and splitting	4.45

The results of Series 2 to 6 are given in Fig. 5. They indicate increases in strength similar to those recorded in Series 1. Series 7 was undertaken to determine the relation between the relative consistency and the increase in strength after a constant period of rest. The results are given in Fig. 9, in which are also included similar data derived from the other series. For Series 7, the 0-day strength was obtained by remolding and again testing the aged specimen immediately after the test to determine its acquired strength. In this way, the influence of the small loss in moisture experienced by the specimens of the other series was entirely eliminated.

All the curves of Fig. 6 indicate that, for a given period of rest, the relative increase in strength increases with the relative consistency of the clay. In other words, in the range of consistencies tested, the rate of hardening increases with the water content. This phenomenon may partly explain the high natural water content of the Laurentian clay. Under the slow process of sedimentation and subsequent consolidation by which the clay strata were formed, if hardening took place in a manner similar to that in the test specimens, the clay would have tended to stabilize itself at the water content for which the rate of hardening was the greatest. This critical water content is evidently

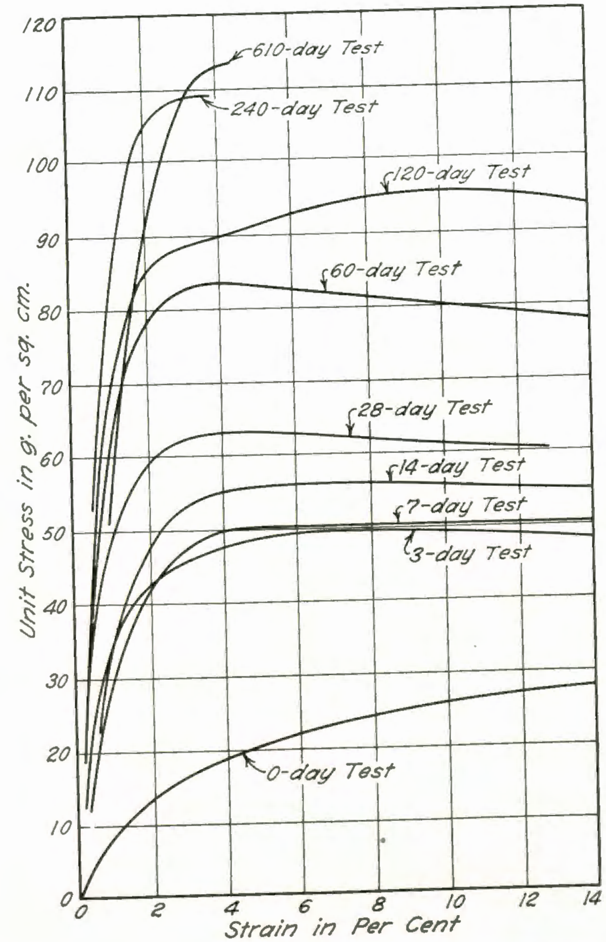


FIG. 4. STRESS-STRAIN CURVES, LAURENTIAN CLAY

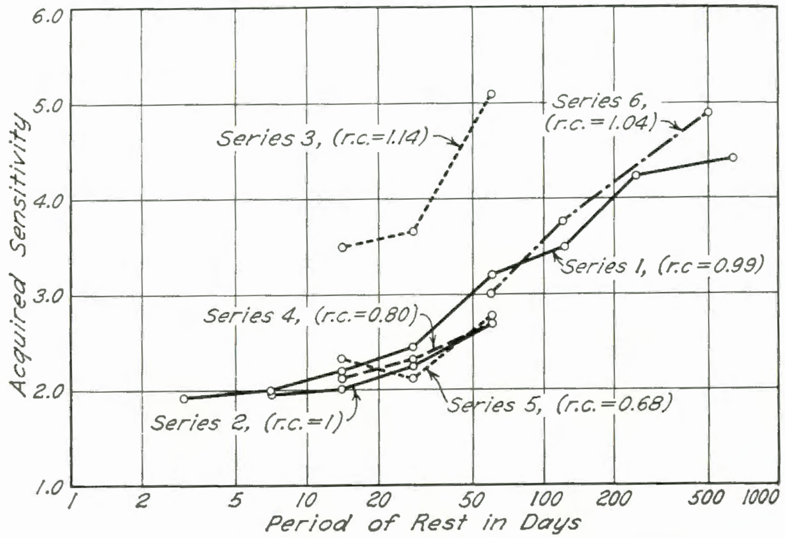


FIG. 5. ACQUIRED SENSITIVITY, LAURENTIAN CLAY, SERIES 2-6

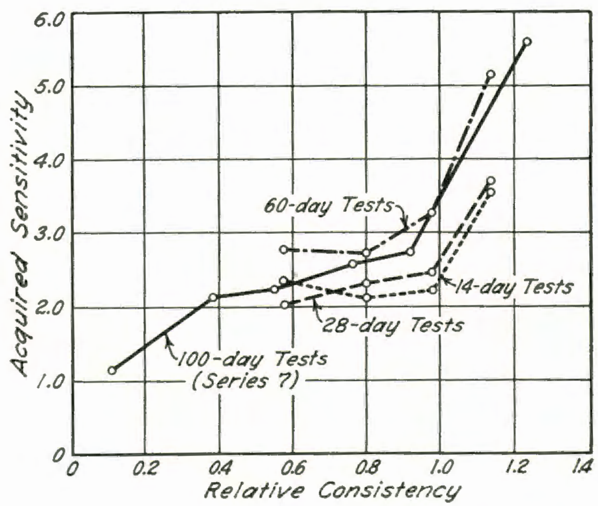


FIG. 6. RELATION OF RATE OF HARDENING AND WATER CONTENT, LAURENTIAN CLAY

higher than any used in this investigation, but the natural water content of the clay is also higher.

b) *Detroit I clay.* — Tests on Detroit I clay comprise four groups designated as Series *a* to *d*. The results are given in Table 3 and Figs. 7-9. Series *a* was the most extensive.

The maximum acquired sensitivity measured in these series of tests reached a value a little over 2.10. Compared with the natural sensitivity of about 2.5 (Table 1) the experiments indicate that this clay may eventually regain a very great part of its original strength lost by remolding.

The results given in Fig. 9 are not extensive enough to permit a definite conclusion, but they seem to indicate that the maximum rate of hardening for the Detroit I clay occurs at a water content near the liquid limit. The water content of the clay in the ground is smaller than this value.

c) *Detroit II clay.* — Only one series of tests was made on this clay, at a relative consistency a little higher than that corresponding to the natural water content of the clay in the ground. The results are given in Table 4 and Figs. 10 and 11.

The rate of increase of both strength and rigidity was slower for this clay than for the other two types already reported. Only after 240 days of rest did the stress-strain curve acquire a shape similar to that of the undisturbed material. The acquired sensitivity, however, reached values up to 2.36, about one third of the natural sensitivity of the undisturbed material.

TABLE 3

UNCONFINED COMPRESSION TESTS ON DETROIT I CLAY: SERIES *a*.
(Relative Consistency = 0.98)

Tested After a Rest Period of (Days) (1)	Water Content, % (2)	Ultimate Strength (for 10% Strain or Less) gm/cm² (3)	Type of Failure (4)	Acquired Sensitivity (5)
0	29.1	34.2	bulge	1.0
3	29.1	49.0	bulge	1.43
7	29.0	51.5	bulge	1.51
14	28.9	49.4	bulge	1.44
28	28.9	58.0	bulge	1.70
60	28.3	68.6	bulge	2.00
120	28.6	64.2	bulge	1.88

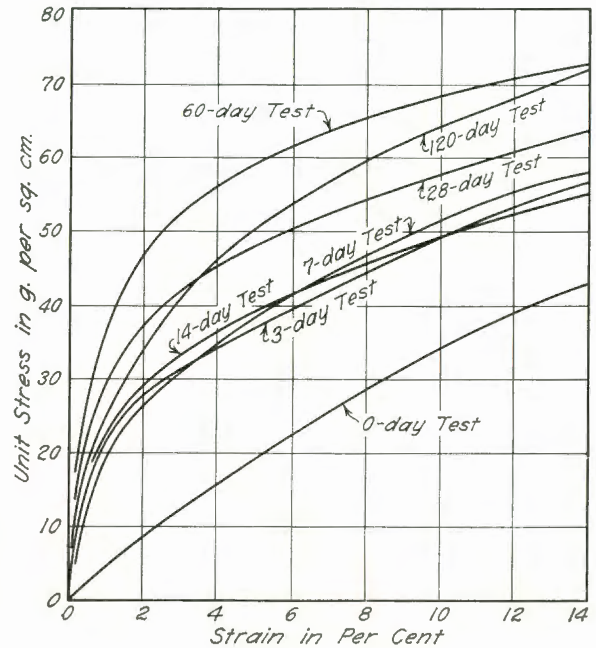


FIG. 7. TESTS ON DETROIT I CLAY

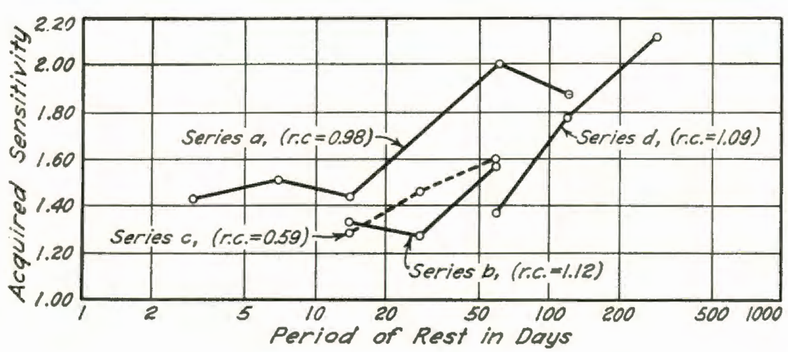


FIG. 8. FURTHER TESTS ON DETROIT I CLAY

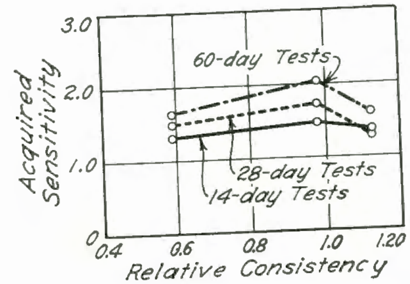


FIG. 9. RESULTS OF TESTS ON DETROIT I CLAY

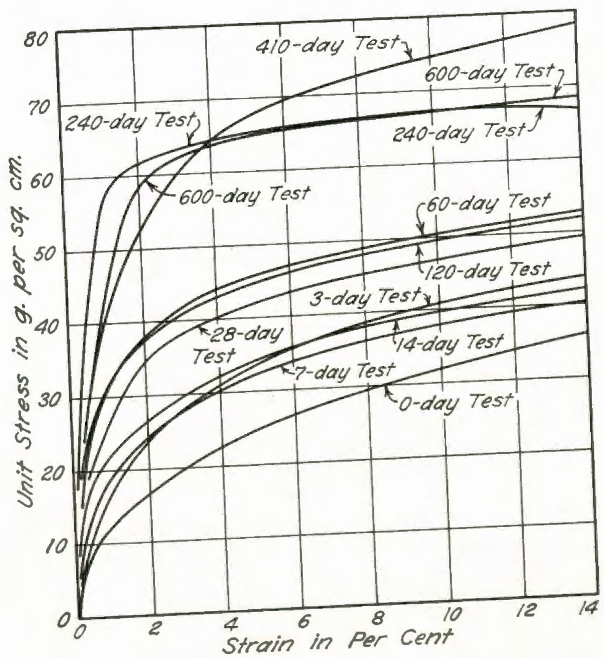


FIG. 10. RESULTS OF TESTS ON DETROIT II CLAY

TABLE 4
UNCONFINED COMPRESSION TESTS ON DETROIT II CLAY
(Relative Consistency = 0.94)

Tested After a Rest Period of (Days) (1)	Water Content, % (2)	Ultimate Strength (for 10% Strain or Less) gm/cm² (3)	Type of Failure (4)	Acquired Sensitivity (5)
0	50.7	31.6	bulge	1.0
3	49.3	40.6	bulge	1.29
7	48.9	37.7	bulge and splitting	1.19
14	50.2	39.6	bulge, shear, and splitting	1.25
28	49.8	46.7	bulge, shear, and splitting	1.48
60	49.6	50.2	bulge and shear	1.59
120	50.3	49.4	bulge and shear	1.56
240	49.9	67.0	bulge and shear	2.12
410	50.1	74.5	bulge and shear	2.36
600	48.9	67.2	shear	2.13

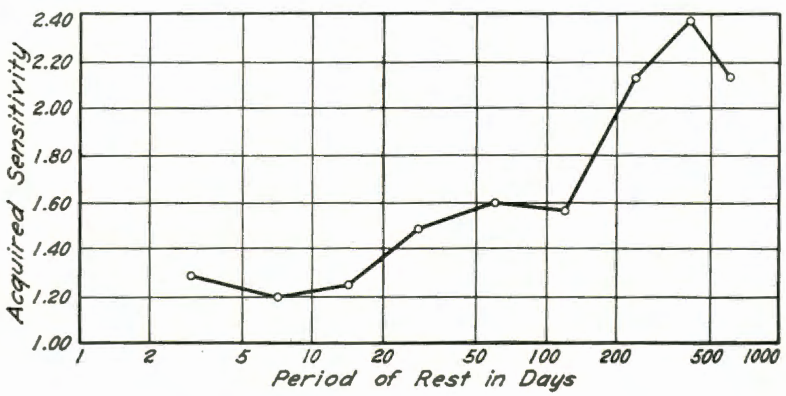


FIG. 11. RESULTS OF TESTS ON DETROIT II CLAY

d) *Mexico clay.*—Tests on this clay comprised a single short series. The results are given in Table 5 and Fig. 12. The increase in strength experienced by the material during the 60-day extent of this series was small when compared with the natural sensitivity of the clay. However, the tests on Detroit II clay suggest that, if allowed to rest longer, the Mexico clay might have developed further increase in its strength and rigidity.

Discussion and Conclusions

The results of the tests indicate that a clay which has lost a great part of its original unconfined compressive strength by a process of complete remolding may regain a sensible part of its lost strength if kept for a certain period at constant water content. Depending on the type of clay, this phenomenon may, in some cases, restore the greater part of the original strength of the undisturbed material. This fact is in contradiction with the theory that assigned most of the difference in strength between undisturbed and remolded clays to a structure that could never be restored once it was destroyed.

Since the test conditions precluded the development of any particular structural arrangement, it appears that the period of rest must have permitted the clay to restore the contacts originally existing between the shells of adsorbed water surrounding each particle. This explanation is more in agreement with the physico-chemical concept of the cause of the strength and rigidity of undisturbed clays. The test results, however, are not in complete accord with this concept. If they were, the thixotropic phenomena should have restored practically all of the original strength of the undisturbed clay. This was

TABLE 5
UNCONFINED COMPRESSION TESTS ON MEXICO CLAY
(Relative Consistency = 1.03)

Tested After a Rest Period of (Days) (1)	Water Content, % (2)	Ultimate Strength (for 10% Strain or Less) gm/cm² (3)	Type of Failure (4)	Acquired Sensitivity (5)
0	61.5	44.4	bulge	1.0
14	....	53.5	bulge	1.32
28	....	55.7	bulge and shear	1.25
60	....	61.8	bulge and shear	1.39

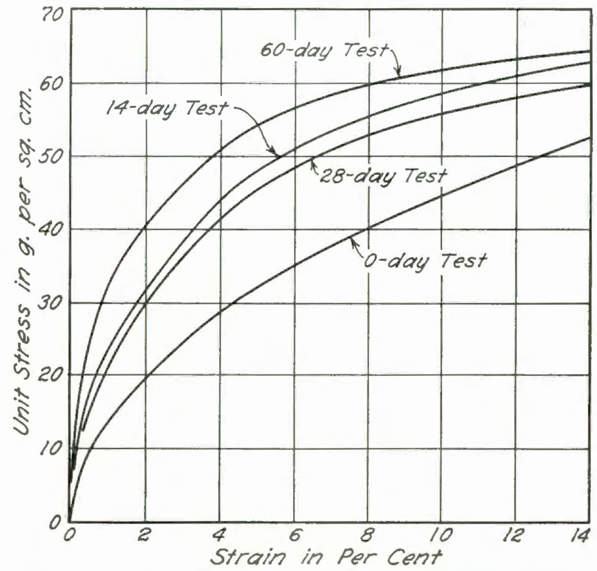


FIG. 12. RESULTS OF TESTS ON MEXICO CLAY

nearly true in the case of the Detroit I clay, but not for the other materials tested. There remains no doubt, however, that a significant part of the strength and rigidity of undisturbed clays depends on the physico-chemical processes resulting from the surface activity of the mineral grains.

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